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Page 195, Footnote 3, Line 3 (omitted), should read "*Roads,* February, 1931."

Page 254, Paragraph 2, Line 2, omit words "by exciting."

Page 254, Paragraph 3, Line 7, should read, "sand, to one in which the sand predominated similarly. Others had a coarse."

Page 255, Paragraph 3, Line 7, for "attained", read "obtained".

Page 256, Paragraph 1, Line 1, for "comprehensive forces", read "compressive forces".

Page 938, Lines 3, 4, and 5, should read:

"Cost of silicon steel erected $= 4 + 0.3 (4 \times 1.34 + 1) = 5.7$ cents.
"Carbon steel, 110 lb @ 5 cents = $5.50 per sq ft.
"Silicon steel, 97 lb @ 5.66 cents = $5.49 per sq ft."

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1858

FORESTS AND STREAM FLOW

By W. G. Hoyt,[1] M. Am. Soc. C. E., and H. C. Troxell,[2]
Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. C. G. Bates, J. E. Willoughby, A. L. Sonderegger, J. C. Stevens, Harry F. Blaney, Daniel W. Mead, Ralph R. Randell, H. K. Barrows, Donald M. Baker, Ralph A. Smead, George H. Cecil, C. W. Sopp, W. P. Rowe, W. C. Lowdermilk, Rhodes E. Rule, Robert E. Kennedy, Herman Stabler, H. S. Gilman, Charles H. Lee, and W. G. Hoyt and H. C. Troxell.

Synopsis

The effect on stream flow of changes in forest and brush cover on specific natural areas is described in this paper. An experiment by the United State Forest Service and the United States Weather Bureau was conducted from 1910 to 1926 on two contiguous tracts of land in Southern Colorado. Data for this study were taken from a report[3] by Messrs. C. G. Bates and A. J. Henry. Stream flow measurements by the United States Geological Survey in co-operation with the State of California and the County of Los Angeles were begun in 1916 on certain areas in California, on some of which accidental denudation by burning afforded opportunity for comparisons not heretofore published. Detailed observations were made in both these areas for several years before changes in cover were accomplished by cutting and by fire, and were continued for several years after such changes.

The effects of the change in vegetable cover on the stream flow characteristics are discussed in this paper and conclusions drawn.

Note.—Presented at the Annual Convention, Yellowstone National Park, July 6, 1932.

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[3] *Monthly Weather Review, Supplement 30.*

INTRODUCTION

In November, 1908, the late H. M. Chittenden, M. Am. Soc. C. E., then Lieutenant-Colonel, Corps of Engineers, U. S. Army, presented a paper[4] before the Society entitled "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers." He summarized the opinions commonly accepted at that time, to the effect that forests exert a beneficial influence on stream flow in the following ways: (1) By storing the waters from rain and melting snow in the bed of humus that develops under forest cover, preventing their rapid rush to the streams and releasing them gradually afterward, thus acting as true reservoirs in equalizing the run-off; (2) by retarding the snow melting in the spring and prolonging the run-off from that source; (3) by increasing precipitation; and, (4) by preventing erosion of the soil on steep slopes and thereby protecting watercourses, canals, reservoirs, and similar works from accumulation of silt.

Colonel Chittenden's study had a negative conclusion. He found that no material influence upon stream flow could be attributed to forests. In its final report,[5] presented January 19, 1916, the Special Committee on Floods and Flood Prevention stated that if reforestation is considered merely from a commercial standpoint, the value to the country of reproducing timber is so great that it obtains general approval, particularly of engineers, who appreciate more than any other class the disadvantages to which the country will be subjected by the destruction of its forests.

Nevertheless, according to the Special Committee, some advocates of reforestation have claimed that forest destruction has increased the height and frequency of floods and diminished the discharge during low water. The basis of those claims for reforestation has been the beneficial influence of forests in preventing floods and improving the navigability of rivers.

This was a moot question in 1916 and the Committee pointed out that even the advocates of reforestation as a means of flood control failed to give any quantitative determination of the effects of forests upon floods. It was stated that, for the engineer to utilize any agency practically, he must know that it will produce a positive effect on which he can rely and which he can measure. The data as to the effect of forestation, soil observation, and kindred methods of flood control were admittedly not susceptible of quantitative analysis at that time.

Expressing his opinion as of 1930, the late Allen Hazen,[6] M. Am. Soc. C. E., called attention to the fact that foresters have urged the importance of forests in controlling and regulating stream flow, increasing the quantities of water available for use, and preventing and reducing flood flows. Although Mr. Hazen felt that the public had generally accepted this point of view, water-works engineers, he declared, have required some proof before they could accept this popular doctrine. Furthermore, according to Mr. Hazen, real evidence of the effect of forests upon stream flow is meager and unsatisfactory, involving the solution of a difficult problem to prove conclusively one way or the other.

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[4] *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 245–546.

[5] *Loc. cit.*, Vol. LXXXI (1917), p. 1218.

[6] "Flood Flows—A Study of Sequences and Magnitude," pp. 152–154, John Wiley & Sons, Inc., 1930.

He admitted that forests undoubtedly affect stream flow in many ways, but that there was no evidence that they modify major flood flows materially. Quoting:

> "There is no reason to think that clearing of land in our country and putting it under cultivation has materially increased the flood quantities over those that existed in previous centuries. Reforesting cleared and burned mountain areas at the head-waters of rivers is advantageous from many aspects, but it is not to be counted on to make a material reduction in floods in rivers flowing from them."

During the twenty years that have elapsed since Colonel Chittenden's paper was written, extensive investigations have been conducted in the Wagonwheel Gap area, in Colorado, for the express purpose of determining the actual effect of forests on stream flow. By chance, also, a forest fire in Southern California destroyed the trees and undergrowth in an area from which records of stream flow had been collected for seven years previously. Information, therefore, is available to show quantitatively the effect on stream flow brought about (*a*) by the complete deforestation of a high mountain area having an annual precipitation of about 20 in. (one-half of which occurs in the form of snow); and (*b*) by the complete denudation of a semi-arid coastal mountain region on which the annual precipitation ranges from 16 to 60 in., with little snow. Conclusive evidence of the effect of forest cover on stream flow from these two widely different areas should be, not only of considerable interest, but also of great value in formulating policies relating to forests and the conservation of water.

It is strange, but nevertheless true, that until the investigations in the Wagonwheel Gap area were completed, no observations had ever been made of the actual effect of deforestation upon stream flow, conducted on a sufficiently large scale and covering a long enough time to insure that the results obtained and the conclusions based on them would not be subject to serious doubt. The investigations near Wagonwheel Gap and in Southern California differ materially from any made heretofore. The observations were made first on the forested areas for a length of time sufficient to determine their natural characteristics; then one of the areas was deforested and one accidentally burned, and observations were continued after the change. In previous investigations, forested and partly deforested areas have been compared only after the change had taken place, and as the regimen of the areas under natural conditions is not known, there is reasonable doubt as to the proper evaluation of the differences in run-off that were observed.

SCOPE OF INVESTIGATIONS

Wagonwheel Gap, Colorado.—Investigations were conducted jointly in the Wagonwheel Gap area, in Colorado, by the United States Forest Service and the United States Weather Bureau, during the period, June 1, 1910, to October 1, 1926.[7] The greatest credit is due to the scientists of the United

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[7] *Monthly Weather Review, Supplement 30,* entitled "Forest and Stream-Flow Experiment at Wagonwheel Gap, Colorado; Final Report on Completion of the Second Phase of the Experiment," by C. G. Bates, Silviculturist, U. S. Forest Service, and A. J. Henry, Meteorologist, U. S. Weather Bureau, 1928.

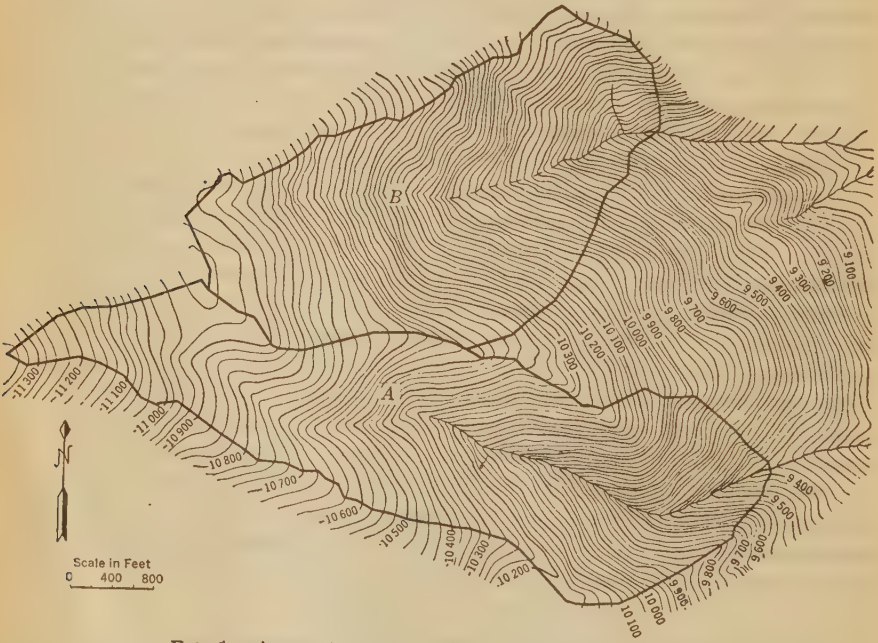
States Department of Agriculture who conceived the project and so successfully carried on detailed observations. The publication of the basic data affords students and investigators the fullest opportunity to make independent analyses.

With this idea in view the writers have made a study of the basic data and have endeavored to present graphically, and in tabular form, in such a way that it may be most easily visualized by engineers and water users, the change in stream flow resulting from deforestation.

Since quantitative measurements of the actual differences in run-off resulting from deforestation and denudation are now available for the first time, the writers feel that the facts should be given far greater weight than conclusions reached by inductive reasoning.

In the following discussion the data collected, both at Wagonwheel Gap and in Southern California, are analyzed to show the effect of deforestation on (1) total run-off and distribution; (2) maximum flow and date of occurrence; (3) summer flow; (4) minimum flow and date of occurrence; and (5) erosion and silt content.

As all the conditions surrounding the Wagonwheel Gap experiment are fully discussed in the report cited, only a general outline of them will be given in this paper. The areas selected (designated for convenience Area *A* and Area *B*) were two contiguous mountain drainage basins in Mineral County, Southern Colorado, in the Rio Grande drainage basin, at altitudes between 9 000 and 11 000 ft. (see Fig. 1). The forest cover in both areas


B
A
11 300
11 200
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N
Scale in Feet
0 400 800

FIG. 1.—AREAS *A* AND *B*, WAGONWHEEL GAP, COLORADO

was representative of the Rocky Mountain region as a whole, and, therefore, other conditions being similar, the results may be reasonably assumed to be representative of many mountain areas throughout the West.

Geologically and topographically both tracts were as similar as it is possible for two small contiguous areas to be, and their combined area (Area *A*, 222.5 acres, and Area *B*, 200.4 acres) was so small that they were both subject to practically identical meteorological conditions. The measurements of run-off and all meteorological observations were made with the greatest possible precision, and the results obtained may be taken at their face value.

The forest cover at the beginning of the investigation was as shown in Table 1.

TABLE 1.—NATURE AND EXTENT OF FOREST COVER IN WAGONWHEEL GAP, COLORADO

Type of cover	PERCENTAGE		Type of cover	PERCENTAGE	
	Area *A*	Area *B*		Area *A*	Area *B*
Burned and not restocked (mostly spruce)	9.5	6.6	Aspen with conifers	14.4	17.1
Barren or rock slide	2.7	3.0	Douglas fir	8.8	11.4
Grass	9.4	6.1	Mainly spruce	11.9	12.0
Aspen without conifers	34.3	43.8	Bristlecone pine (open)	9.0	....
Total				100.0	100.0

During the summer of 1919 Area *B* was deforested, all tree growth being cut except in a narrow strip along the course of the stream, which was not cut until 1920. The slash from the larger green conifers and the tops and stems of the smaller evergreens and aspens were burned in 1921. After the deforestation there was a growth of grass, herbs, and aspen, and, in places, the aspen sprouts had reached heights of 3 to 6 ft. at the end of the 7-year period. In general, the grasses were reproduced on south exposures and grass and herbs on north exposures. Messrs. Bates and Henry have used the term "denudation" to describe the removal of the trees from the area. Webster defines denudation as "act of denuding; * * * the laying bare of rocks by the removal of overlying material; erosion." The writers believe that the term "deforestation" more correctly defines the change that occurred on Area *B*, because only in isolated places where slash was burned (comprising a very small percentage of the whole), was there actual destruction of soil covering. "Denudation," however, correctly describes the change that occurred in the Southern California area.

Reliable observations at Wagonwheel Gap were made for eight years in the natural state and for seven years after deforestation. (There is some doubt as to the accuracy of the run-off records prior to August, 1911, so that the complete record runs from September, 1911, to September, 1926.) This study is based necessarily on the hypothesis that during the first eight years the run-off relations and differences between the two areas were so well indicated that the results obtained can be safely used as indices of the changes in run-off resulting from complete deforestation.

Many of the pertinent annual and monthly climatological data for the two areas are given in the accompanying tables. Briefly, the mean annual temperature was about 34° Fahr., the range being from 80° Fahr. during June, July, and August, to temperatures below zero occasionally during all months from October to March. The mean annual precipitation was about 21 in., one-half of which occurred in the form of snow. The climatic conditions were very similar during both periods.

The mean annual temperature of Area *A* was 34.0° Fahr., during both periods. The mean annual temperature of Area *B* was 1.3° warmer after deforestation than before. The mean annual precipitation over Area *A* was 21.03 in. during the first period and 21.16 in. during the second period; over Area *B*, the mean annual precipitation was 21.10 and 20.83 in., respectively, during the first and second periods. The accumulated precipitation on these two areas generally agrees within 0.5%, or less than the probable error of the observations. In the accompanying tables observations of run-off are given, in inches, over the drainage basin, corresponding to the base data given in the U. S. Weather Bureau report, and, also, in second-feet per square mile.

Southern California Area.—On August 31, 1924, a forest fire started in the San Gabriel Canyon, north of Azusa, Calif. The fire spread rapidly and had soon burned over some of the drainage areas tributary to the San Gabriel River (see stipled area in Fig. 2). Among these areas were those of Fish, Rogers, and Sawpit Creeks. The fire did not reach the drainage basin of Santa Anita Creek. On each of these creeks the U. S. Geological Survey had been operating gauging stations for several years, in co-operation with the California State Engineer and Los Angeles County. Each station was equipped with a water-stage recorder mounted in a concrete house. Measurements of discharge were made with a current meter, and the records were kept and published in the usual manner.

Of the three burned areas that of Fish Creek offers the best discharge record, in that there are no diversions from the stream nor artificial regulation above the gauging station. On Rogers Creek there are two diversions above the gauging station. No record has been kept of the quantity diverted, but although the structures are relatively small, they divert the entire flow during the summer. On Sawpit Creek the City of Monrovia diverts water above the gauging station. The city has a series of tunnels in the head-waters, which is one of the main sources of the flow in the Monrovia pipe line.

For these reasons Fish Creek was selected for the study. To establish the effect of the fire on the discharge of this creek, it was necessary to determine the relation between its flow and that of Santa Anita Creek for the period prior to the fire.

The canyons of both Fish and Santa Anita Creeks are well defined by mountain divides and are separated only by the basin of Sawpit Creek. Both streams originate at altitudes greater than 4 000 ft. The east side of Mount Wilson drains into Santa Anita Creek. Monrovia Hill, a peak of 5 267 ft., separates the head-waters of the two streams, Fish Creek draining the east side and Santa Anita Creek the west. Santa Anita Canyon is fan-shaped; Fish Canyon is oblong; and the side walls of both canyons are steep and rugged. The drainage area above the gauging station on Fish Creek is

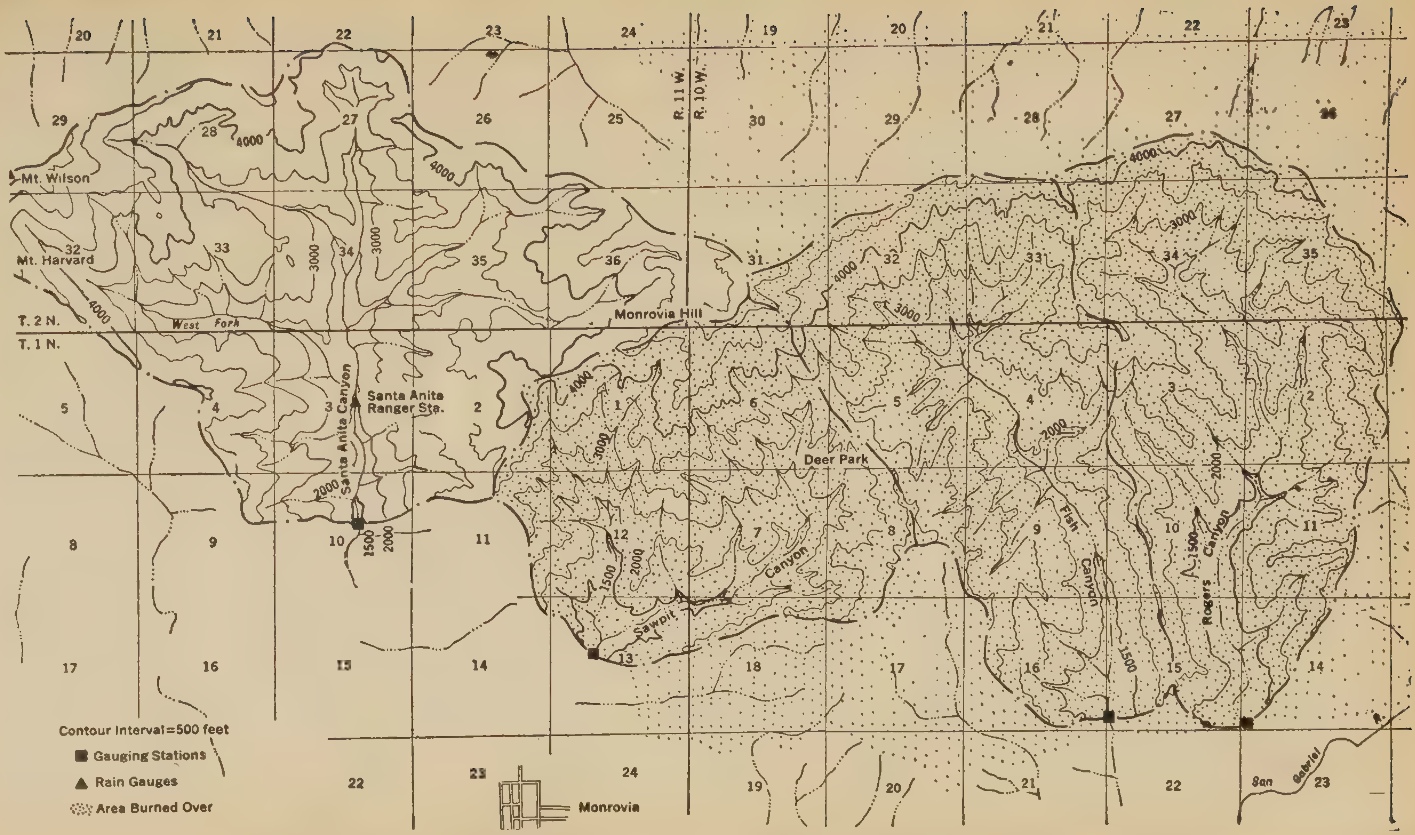
Mt. Wilson
Mt. Harvard
West Fork
T. 2 N.
T. 1 N.
R. 11 W.
R. 10 W.
Santa Anita Canyon
Santa Anita Ranger Sta.
Monrovia Hill
Deer Park
Sawpit
Canyon
Fish
Canyon
Rogers
Canyon
San Gabriel
Monrovia
4000
3000
2000
1500
Contour Interval=500 feet
■ Gauging Stations
▲ Rain Gauges
Area Burned Over

FIG. 2.—SANTA ANITA, FISH, ROGERS, AND SAWPIT CREEKS, IN THE SOUTHERN PART OF CALIFORNIA

6.5 sq. miles, and that of Santa Anita Creek, 10.5 sq. miles. The vegetative cover consisted of a heavy brush growth of sumac (*Rhus sp.*) and mountain mahogany (*Cercocarpus montanus*). Interspersed between the brush growth were much smaller plants, such as white sage (*Artemisia californica*), grass, and yucca (*Yucca radiosa*). In sheltered tracts and on the north sides of many of the numerous small tributary canyons grew small groups of oak. In the bottoms of the canyons along the streams were large numbers of sycamores and alders. The areas are in the Angeles National Forest, but they support no timber, and neither area was used for grazing. Their principal value was for recreation.

The fire in the summer of 1924 completely stripped the canyon walls of Fish Creek of all plant life and tree litter. Some of the alders and sycamores adjacent to the stream escaped burning through the efforts of the fire fighters. After the first rains in the following spring a different plant association comprising sage, wild flowers, and grasses developed, which tended to cover the soil left bare by the fire. By the fall of 1930 little evidence of the fire remained except that possibly larger areas of white sage and small plants grew in.

The gauging stations on both streams are ideally located. The station on Santa Anita Creek is in a rock gorge and the crest of small waterfalls forms the control. On Fish Creek, the control is an outcrop of bed-rock, which was smoothed up by building a small concrete dam on it. It is doubtful if any underflow passes either station. There are no diversions above either station. The stream-flow data prior to 1927 have been published by the U. S. Geological Survey,[8] and later data are taken from unpublished records in the Survey files.

The best rainfall records in this general region are those of the Mount Wilson and Santa Anita Ranger Stations. Both stations are operated and the records published by the U. S. Weather Bureau. As it was not anticipated that a study of denudation would be made in either area, detailed climatological and other records, such as those which were collected with great care and accuracy in the Wagonwheel Gap area, are lacking for this study.

No records of temperature were collected directly in the Southern California area during the first or second period of the study. At Mount Wilson, on the western divide of Santa Anita Creek, the average annual temperature during the two periods was 55.7° Fahr., and the daily temperature ranged from about 100° Fahr. in summer to less than 32° Fahr. in winter.

EFFECT OF DEFORESTATION AND DENUDATION

Annual Run-Off.—In the following tables the period of record before deforestation and denudation is designated the "first period," and the period of record after deforestation and denudation, the "second period." The precipitation given for the Wagonwheel Gap area (Table 2) is the average of Area *A* and Area *B*; that for the Southern California area is the precipitation recorded at Mount Wilson.

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[8] *Water Supply Papers Nos. 481, 511, 531, 551, 571, 591, 611,* and *631,* U. S. Geological Survey.

In determining the effect of deforestation and denudation on these areas it was first necessary to ascertain what the run-off of Area *B* and the Fish Creek Basin would have been if conditions of vegetative cover had not been changed by deforestation or denudation. Although the two contiguous areas, *A* and *B* at Wagonwheel Gap, appeared to be alike in many respects, their run-off relations varied greatly. For example, Area *B* before deforestation had a daily run-off as low as 50% of that of Area *A* during the rising flood stages and as high as 150% during the falling stages. (See Fig. 3.) The relation varied also during the remainder of the year, depending to a considerable extent on the magnitude of the daily discharge and the season. Variations in discharge relations also occurred in the Southern California area on Fish and Santa Anita Creeks.

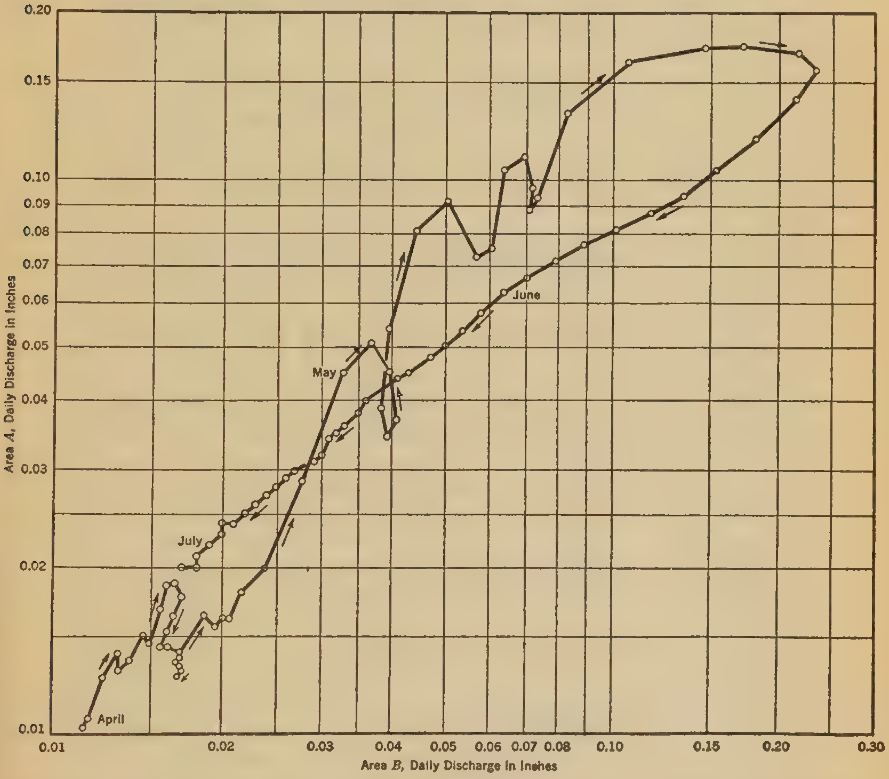

FIG. 3.—RELATION OF DAILY DISCHARGE, AREAS *A* AND *B* (WAGONWHEEL GAP), DURING RISING AND FALLING STAGES, APRIL 1 TO JULY 5, 1912

In view of these variations, the writers believed that it was desirable to compute the probable daily discharge for the period after deforestation or denudation rather than to rely on monthly figures. This permitted a much more thorough analysis of the available data.

The normal daily discharge on Area *B* was determined by comparisons established prior to deforestation, using the following methods: Beginning in October, 1919, with the falling stage on Area *B* and continuing through July,

TABLE 2.—MONTHLY PRECIPITATION AND RUN-OFF, IN INCHES, DURING THE FIRST PERIOD*

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
WAGONWHEEL GAP AREA													
1911–1912:													
Precipitation	4.54	1.15	1.50	0.35	0.40	3.06	1.54	0.34	2.17	4.18	1.74	0.43	21.40
Run-off: Area *A*	0.901	0.456	0.367	0.312	0.267	0.308	0.460	2.951	1.044	0.566	0.413	0.323	8.370
Run-off: Area *B*	0.984	0.433	0.362	0.337	0.307	0.338	0.504	2.928	0.962	0.497	0.384	0.335	8.371
1912–1913:													
Precipitation	2.56	0.44	0.70	1.05	1.09	1.48	0.82	0.52	3.10	2.40	2.40	2.60	19.16
Run-off: Area *A*	0.339	0.300	0.286	0.278	0.227	0.284	0.795	0.802	0.586	0.356	0.264	0.261	4.779
Run-off: Area *B*	0.380	0.350	0.339	0.329	0.289	0.323	0.564	1.020	0.690	0.378	0.274	0.278	5.214
1913–1914:													
Precipitation	0.97	1.85	2.37	2.20	0.66	0.74	0.94	2.27	1.63	5.06	2.18	1.38	22.25
Run-off: Area *A*	0.287	0.255	0.249	0.241	0.216	0.271	0.545	1.639	0.769	0.491	0.362	0.304	5.628
Run-off: Area *B*	0.307	0.289	0.288	0.281	0.247	0.302	0.462	1.734	0.694	0.380	0.290	0.277	5.553
1914–1915:													
Precipitation	2.22	0.02	1.20	0.90	2.40	0.35	3.74	1.54	0.50	2.25	1.86	2.94	19.92
Run-off: Area *A*	0.308	0.238	0.226	0.259	0.218	0.240	0.460	1.621	0.835	0.402	0.294	0.253	5.354
Run-off: Area *B*	0.315	0.291	0.287	0.297	0.261	0.285	0.436	1.584	0.810	0.337	0.258	0.247	5.407
1915–1916:													
Precipitation	0.36	1.98	2.72	3.48	0.49	1.60	1.99	0.34	0.09	5.21	3.14	1.52	22.92
Run-off: Area *A*	0.274	0.245	0.226	0.219	0.207	0.365	0.652	1.721	0.604	0.407	0.373	0.303	5.593
Run-off: Area *B*	0.281	0.271	0.268	0.269	0.248	0.373	0.570	1.784	0.555	0.325	0.323	0.289	5.555
1916–1917:													
Precipitation	4.37	0.20	1.48	1.89	0.74	1.58	4.98	2.27	0.13	1.94	2.08	1.17	22.83
Run-off: Area *A*	0.424	0.341	0.254	0.236	0.211	0.249	0.610	3.258	2.622	0.700	0.419	0.320	9.643
Run-off: Area *B*	0.457	0.382	0.322	0.303	0.266	0.295	0.580	2,962	3.097	0.550	0.341	0.287	9.843
1917–1918:													
Precipitation	0.19	1.07	0.27	1.29	1.92	2.08	0.72	0.14	1.04	3.84	3.20	3.12	18.88
Run-off: Area *A*	0.317	0.271	0.253	0.233	0.199	0.222	0.315	0.420	0.288	0.236	0.201	0.240	3.195
Run-off: Area *B*	0.321	0.313	0.311	0.303	0.268	0.316	0.336	0.372	0.283	0.248	0.215	0.250	3.534
1918–1919:													
Precipitation	1.05	2.52	1.88	0.06	1.80	2.58	2.10	1.32	0.89	4.37	1.04	1.53	21.14
Run-off: Area *A*	0.238	0.222	0.223	0.217	0.191	0.223	0.782	2.138	0.803	0.480	0.309	0.255	6.079
Run-off: Area *B*	0.252	0.241	0.237	0.233	0.206	0.237	0.681	2.284	0.677	0.362	0.289	0.270	5.970

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* All figures in this table, including totals and averages, have been rounded off from the 5-place data.

TABLE 2.—(*Continued*)

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
SOUTHERN CALIFORNIA AREA													
1917–1918:													
Precipitation	0	0.45	0.05	0.64	13.39	12.85	0.37	0.31	0.07	0.40	0.39	1.90	30.82
Run-off: Santa Anita Creek	0.07	0.08	0.08	0.09	0.80	3.97	0.75	0.40	0.21	0.10	0.07	0.06	6.68
Run-off: Fish Creek	0.04	0.09	0.09	0.10	0.99	5.93	0.73	0.35	0.15	0.03	0.02	0.03	8.55
1918–1919:													
Precipitation	0.58	4.79	2.53	1.12	4.05	3.96	0.74	0.16	0	Trace	Trace	4.05	21.98
Run-off: Santa Anita Creek	0.07	0.13	0.20	0.13	0.28	0.34	0.19	0.13	0.06	0.02	0.01	0.02	1.58
Run-off: Fish Creek	0.05	0.13	0.24	0.17	0.34	0.52	0.23	0.15	0.03	0	0	0.01	1.87
1919–1920:													
Precipitation	1.37	1.72	4.36	0.84	6.95	10.12	1.70	0.08	0	0	0	0.03	27.17
Run-off: Santa Anita Creek	0.06	0.08	0.27	0.12	0.45	1.87	0.82	0.33	0.16	0.06	0.03	0.03	4.28
Run-off: Fish Creek	0.04	0.09	0.37	0.14	0.80	3.10	1.04	0.43	0.18	0.02	0	0	6.21
1920–1921:													
Precipitation	2.25	2.08	1.73	5.66	1.44	8.77	1.06	11.04	0	0.03	0	0.22	34.28
Run-off: Santa Anita Creek	0.05	0.07	0.11	0.31	0.21	1.29	0.28	1.46	0.49	0.19	0.08	0.04	4.58
Run-off: Fish Creek	0.03	0.08	0.12	0.40	0.16	1.94	0.28	1.34	0.42	0.06	0.02	0.01	4.86
1921–1922:													
Precipitation	1.12	0.11	29.40	12.86	9.90	3.76	1.91	1.20	0	0	0	0	60.26
Run-off: Santa Anita Creek	0.07	0.05	7.59	5.00	7.83	3.99	2.15	1.37	0.75	0.40	0.27	0.17	29.64
Run-off: Fish Creek	0.02	0.03	5.82	4.47	8.03	3.29	1.85	1.27	0.64	0.30	0.12	0.06	25.90
1922–1923:													
Precipitation	0.55	4.39	9.05	3.33	1.39	1.50	3.40	0	0.18	0.02	0	0.67	24.48
Run-off: Santa Anita Creek	0.17	0.37	1.35	0.48	0.54	0.38	0.42	0.27	0.18	0.12	0.06	0.04	4.38
Run-off: Fish Creek	0.09	0.31	1.64	0.53	0.64	0.38	0.47	0.17	0.08	0.02	0.02	0.02	4.37
1923–1924:													
Precipitation	0.68	0.36	1.03	1.86	0.30	9.26	3.07	0.01	0	0	0	0	16.57
Run-off: Santa Anita Creek	0.05	0.09	0.10	0.12	0.10	0.33	0.26	0.13	0.05	0.01	0.01	0.01	1.26
Run-off: Fish Creek	0.03	0.04	0.07	0.09	0.07	0.43	0.20	0.06	0.01	0	0	0	1.00

the daily discharge on Area *A* was plotted against that of Area *B*. It was found by inspection that the relation depended to a considerable extent on the quantity of the run-off. With this in mind, the records were divided into three groups: That of 1912 and 1917, which represented the years of greatest run-off; that of 1914, 1915, 1916, and 1919, the medium years; and that of 1913 and 1918, the years of low run-off. The daily relation was plotted for each of these groups on separate sheets. A curve was then drawn averaging all the points plotted. Fig. 4 is a typical curve of relation between the run-off of Area *A* and Area *B* during the falling stages of May, June, and July in the years of greatest run-off, and Fig. 3 represents the typical daily relation during rising and falling stages. Curves were likewise drawn showing the relations that existed during August, September, October, November, December to March, and April and May.

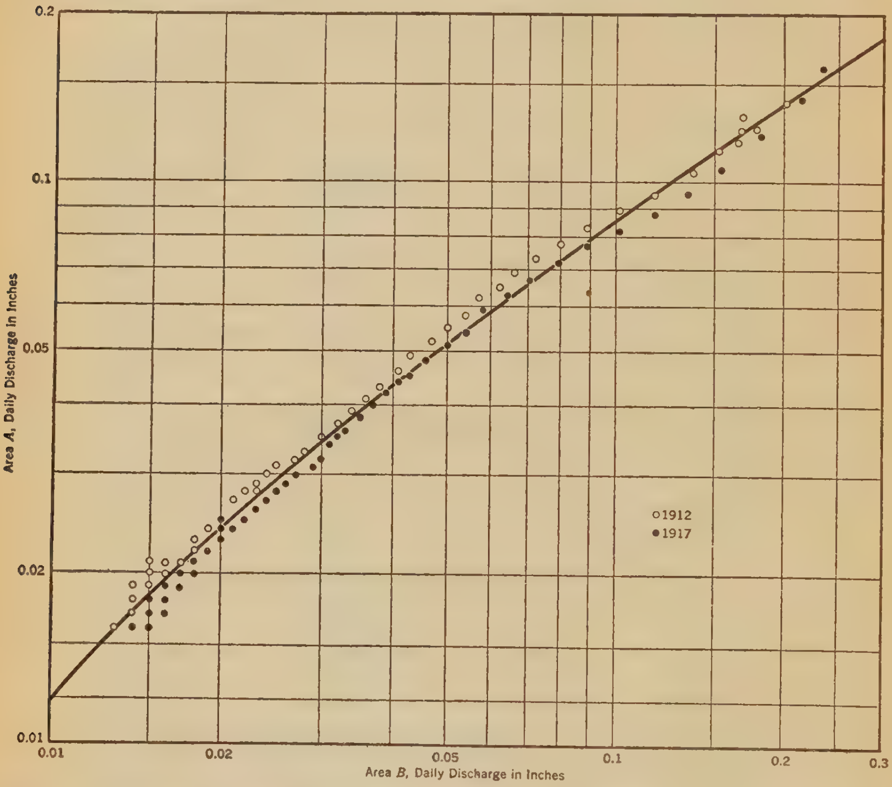

FIG. 4.—RELATION OF DAILY DISCHARGE, AREAS *A* AND *B* (WAGONWHEEL GAP), DURING FALLING STAGES, MAY TO JULY, 1912 AND 1917

The normal daily discharge in Area *B* for the period after deforestation was assumed to be the discharge obtained by applying the observed daily discharge of Area *A* to the appropriate curves, with one exception. It was noted that prior to the deforestation about three days were usually required for the change from the rising stage curve of April and May to the falling stage curve of May to July. During this three-day transition period it was necessary to make an interpolation for Area *B* based on the record from Area *A*.

For the Southern California area, curves were drawn showing the relations, prior to the fire, for January to March, April, May, June, July, August, September, and October to December. In plotting curves the observations made during 1921-22 were not used, as in that year considerable of the precipitation occurred in the form of snow, the run-off from which represented an entirely different relation. None of the seasons following denudation on Fish Creek was comparable to 1921-22.

Table 3 shows the monthly data for the second period and, in addition, the computed normal run-off, based on the relations previously described, of Area *B* and Fish Creek as it would have been had there been no change in vegetative cover. The difference between the actual and the computed normal run-off is also shown in Table 3.

The values in the last column of Table 3 show that the maximum gain on Area *B* occurred the third year after deforestation and amounted to 2.141 in., or 32.3%, and that there was a gradual decrease for the remainder of the period. On Fish Creek the maximum gain occurred the second year after the fire and amounted to 3.12 in., or 26 per cent. The gain on Fish Creek the first year after the fire was 2.47 in., or 231 per cent.

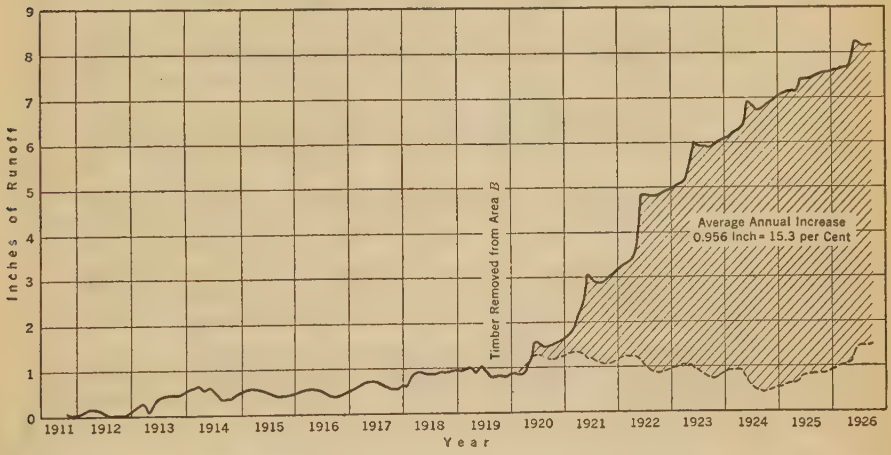

FIG. 5.—ACCUMULATED EXCESS OF RUN-OFF FOR AREA *B* OVER AREA *A*, WAGONWHEEL GAP, COLORADO

Figs. 5 and 6 show the accumulated difference in run-off of Areas *A* and *B* and of Fish and Santa Anita Creeks during the first and second periods. The cross-hatched area in Fig. 5 represents an average annual increase of 0.956 in., or 15.3%, from Area *B* during the first seven years after complete deforestation. The dashed line from 1920 to 1926 represents the normal

TABLE 3.—MONTHLY PRECIPITATION AND RUN-OFF, IN INCHES, AND CHANGE IN RUN-OFF DURING THE SECOND PERIOD*

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
WAGONWHEEL GAP AREA													
1919–1920: Precipitation	1.98	4.82	0.82	0.57	1.49	1.91	2.16	2.45	1.13	1.96	1.06	1.78	22.14
Run-off: Area *A*	0.276	0.254	0.250	0.237	0.218	0.241	0.347	3.583	1.292	0.503	0.357	0.305	7.865
Run-off: Area *B*:													
Actual	0.285	0.270	0.273	0.263	0.243	0.295	0.555	4.010	1.235	0.451	0.340	0.330	8.550
Normal	0.286	0.287	0.293	0.281	0.261	0.286	0.359	3.887	1.299	0.414	0.312	0.288	8.253
Increase,† in inches	—0.001	—0.017	—0.020	—0.018	—0.018	+0.009	+0.196	+0.123	—0.064	+0.037	+0.028	+0.042	+0.297
Increase, percentage	—0.3	—5.9	—6.8	—6.4	—6.9	+3.2	+54.6	+3.2	—4.9	+8.9	+9.0	+14.6	+3.6
1920–1921: Precipitation	3.42	1.88	1.52	1.36	0.60	1.08	1.58	2.51	1.75	3.28	3.05	0.55	22.59
Run-off: Area *A*	0.316	0.291	0.278	0.266	0.240	0.339	0.567	2.240	1.083	0.534	0.416	0.328	6.898
Run-off: Area *B*:													
Actual	0.364	0.360	0.351	0.336	0.318	0.703	0.737	2.927	0.977	0.493	0.397	0.364	8.328
Normal	0.326	0.327	0.314	0.307	0.277	0.367	0.491	2.130	1.157	0.443	0.353	0.306	6.798
Increase†, in inches	+0.038	+0.033	+0.037	+0.029	+0.041	+0.336	+0.246	+0.797	—0.180	+0.050	+0.044	+0.058	+1.529
Increase, percentage	+11.7	+11.0	+11.8	+9.5	+14.8	+91.5	+50.1	+37.4	—15.5	+11.3	+12.5	+18.9	+22.5
1921–1922: Precipitation	0.80	0.32	1.50	2.95	1.50	2.72	1.65	2.47	1.20	1.77	3.37	0.74	20.98
Run-off: Area *A*	0.324	0.292	0.282	0.262	0.237	0.284	0.457	2.549	1.001	0.447	0.396	0.298	6.830
Run-off: Area *B*:													
Actual	0.397	0.366	0.346	0.348	0.311	0.377	0.862	3.641	0.996	0.428	0.379	0.317	8.769
Normal	0.336	0.325	0.318	0.301	0.274	0.323	0.429	2.416	0.910	0.373	0.340	0.282	6.627
Increase†, in inches	+0.061	+0.041	+0.028	+0.047	+0.034	+0.054	+0.433	+1.225	+0.086	+0.055	+0.039	+0.035	+2.141
Increase, percentage	+18.1	+12.6	+8.8	+15.6	+12.4	+16.7	+10.1	+50.8	+9.4	+14.7	+11.5	+12.4	+32.3
1922–1923: Precipitation	0.64	4.81	1.16	0.79	0.94	1.68	2.08	0.66	1.00	2.52	4.04	3.75	24.08
Run-off: Area *A*	0.316	0.300	0.294	0.277	0.245	0.272	0.446	2.064	0.738	0.427	0.366	0.345	6.091
Run-off: Area *B*:													
Actual	0.350	0.349	0.348	0.325	0.289	0.334	0.698	2.680	0.697	0.382	0.351	0.363	7.168
Normal	0.328	0.332	0.328	0.315	0.282	0.311	0.419	1.991	0.673	0.357	0.318	0.317	5.971
Increase†, in inches	+0.022	+0.017	+0.020	+0.010	+0.007	+0.023	+0.279	+0.689	+0.024	+0.025	+0.033	+0.046	+1.195
Increase, percentage	+6.7	+5.1	+6.1	+2.9	+2.5	+7.4	+66.6	+34.6	+35.7	+7.0	+10.4	+14.5	+20.0
1923–1924: Precipitation	2.60	1.67	1.34	0.60	1.06	3.15	1.44	0.72	0.17	1.87	1.16	1.16	16.86
Run-off: Area *A*	0.388	0.348	0.314	0.282	0.257	0.278	0.920	2.518	0.789	0.428	0.305	0.276	7.104
Run-off: Area *B*:													
Actual	0.453	0.411	0.358	0.340	0.316	0.353	1.006	3.093	0.646	0.391	0.316	0.332	8.016
Normal	0.411	0.401	0.346	0.319	0.293	0.317	0.754	2.396	0.675	0.359	0.276	0.264	6.811
Increase†, in inches	+0.042	+0.010	+0.012	+0.021	+0.023	+0.036	+0.252	+0.697	—0.029	+0.032	+0.40	+0.068	+1.204
Increase, percentage	+10.2	+2.5	+3.5	+6.6	+7.9	+11.4	+33.4	+29.2	—4.3	+8.9	+14.5	+25.8	+17.7
1924–1925: Precipitation	1.48	1.06	2.16	0.16	0.58	2.92	1.46	1.18	2.16	3.82	3.70	1.46	22.15
Run-off: Area *A*	0.316	0.289	0.288	0.274	0.244	0.294	0.599	0.696	0.440	0.308	0.267	0.254	4.269
Run-off: Area *B*:													
Actual	0.382	0.351	0.345	0.324	0.294	0.355	0.588	0.934	0.447	0.338	0.308	0.283	4.948
Normal	0.327	0.322	0.322	0.313	0.280	0.330	0.590	0.830	0.488	0.314	0.275	0.248	4.639
Increase†, in inches	+0.055	+0.029	+0.023	+0.011	+0.014	+0.025	—0.062	+0.104	—0.041	+0.024	+0.033	+0.035	+0.310
Increase, percentage	+16.8	+9.0	+7.1	+3.5	+5.0	+7.5	—0.4	+12.5	—8.4	+7.6	+12.0	+14.1	+6.69

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* All figures in this table, including totals and averages, have been rounded off from the 5-place data.

† Negative sign denotes decrease.

TABLE 3.—(*Continued*)

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
WAGONWHEEL GAP AREA — (*Continued*).													
1925–1926: Precipitation	2.30	1.66	0.12	0.68	0.38	2.17	1.47	1.91	0.95	2.55	2.28	1.70	18.16
Run-off: Area *A*	0.279	0.252	0.247	0.231	0.208	0.236	0.444	1.068	0.588	0.352	0.260	0.212	4.379
Run-off: Area *B*:													
Actual	0.310	0.286	0.274	0.258	0.236	0.291	0.442	1.612	0.540	0.320	0.254	0.228	5.050
Normal	0.290	0.286	0.292	0.277	0.250	0.282	0.416	1.327	0.765	0.367	0.271	0.218	5.032
Increase†, in inches	+0.020	0.000	—0.018	—0.019	—0.014	+0.009	+0.026	+0.285	—0.216	—0.047	—0.017	+0.010	+0.019
Increase, percentage	+6.9	......	—6.2	—6.9	—5.6	+0.3	+6.3	+21.5	-28.5	-12.8	—6.3	+4.6	+0.04
SOUTHERN CALIFORNIA AREA													
1924–1925: Precipitation:													
Mount Wilson	1.06	2.74	2.89	0.33	2.77	3.60	6.16	0.97	1.39	0	0	0	21.91
Santa Anita Ranger Station	1.80	3.05	2.86	0.32	1.89	2.62	6.54	1.23	0.84	0	0	0	21.15
Run-off:													
Santa Anita Creek	0.03	0.07	0.11	0.09	0.10	0.11	0.45	0.14	0.10	0.02	0.01	0.01	1.24
Fish Creek:													
Actual	0.01	0.10	0.26	0.10	0.14	0.50	1.92	0.24	0.16	0.06	0.03	0.02	3.54
Normal	0.01	0.05	0.12	0.07	0.07	0.09	0.51	0.10	0.05	0	0	0	1.07
Increase†, in inches	0	+0.05	+0.14	+0.03	+0.07	+0.41	+1.41	+0.14	+0.11	+0.06	†0.03	+0.02	+2.47
Increase, percentage	∞	+100	+117	+43	+100	+456	+276	+140	+220	∞	∞	∞	+231
1925–1926: Precipitation:													
Mount Wilson	1.65	2.72	2.95	2.66	7.78	0.64	17.55	0.31	0	0	0	0	36.26
Santa Anita Ranger Station	2.88	1.78	3.87	2.46	8.11	0.52	18.92	1.06	0	0	0	0	39.60
Run-off:													
Santa Anita Creek	0.02	0.05	0.09	0.09	0.66	0.13	5.70	0.62	0.23	0.10	0.05	0.03	7.77
Fish Creek:													
Actual	0.05	0.06	0.24	0.16	1.96	0.39	10.85	0.63	0.29	0.13	0.09	0.07	14.92
Normal	0.01	0.03	0.08	0.08	0.83	0.12	9.76	0.66	0.19	0.03	0.01	0	11.80
Increases†, in inches	+0.04	+0.03	+0.16	+0.08	+1.13	+0.27	+1.09	—0.03	+0.10	+0.10	+0.08	+0.07	+3.12
Increase, percentage	+400	+100	+200	+100	+136	+225	+11.2	—4.5	+52.6	+334	+800	∞	+26
1926–1927: Precipitation:													
Mount Wilson	0.10	5.53	2.97	1.84	17.07	4.16	1.56	0.14	0	0	0	0	33.37
Santa Anita Ranger Station	0.11	6.00	3.17	2.72	19.77	†	†	0.80	0.15	0	0	0	....
Run-off:													
Santa Anita Creek	0.03	0.21	0.28	0.26	4.43	1.90	1.02	0.50	0.31	0.16	0.08	0.04	9.24
Fish Creek:													
Actual	0.09	0.44	0.34	0.46	8.36	2.34	1.11	0.63	0.41	0.21	0.14	0.12	14.65
Normal	0.01	0.23	0.31	0.33	7.62	2.39	1.25	0.54	0.27	0.07	0.03	0.02	13.07
Increases‡, in inches	+0.08	+0.21	+0.03	+0.13	+0.74	—0.05	—0.14	+0.09	+0.14	+0.14	+0.11	+0.10	+1.58
Increase, percentage	+800	+91	+9.7	+39.4	+9.7	—2.1	—11.2	+16.7	+51.9	+200	+367	+500	+12.1

†Negative sign denotes decrease.
‡Incomplete.

TABLE 3.—(*Continued*)

SOUTHERN CALIFORNIA AREA—(*Continued*).

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
1927–1928: Precipitation:													
Mount Wilson	3.13	1.18	5.85	0.30	4.35	3.02	0.88	1.07	0	0	0	0	19.78
Santa Anita Ranger Station	2.94	1.34	†	†	3.37	3.32	1.42	1.10	0	0	0	0	....
Run-off:													
Santa Anita Creek	0.05	0.14	0.27	0.20	0.37	0.31	0.21	0.14	0.06	0.02	0.01	0.01	1.79
Fish Creek:													
Actual	0.13	0.27	0.48	0.28	0.47	0.31	0.24	0.17	0.07	0.03	0.01	0.01	2.47
Normal	0.02	0.15	0.32	0.23	0.41	0.33	0.20	0.11	0.02	0	0	0	1.79
Increase‡, in inches	+0.11	+0.12	+0.16	+0.05	+0.06	−0.02	+0.04	+0.06	+0.05	+0.03	+0.01	+0.01	+0.68
Increase, percentage	+550	+80	+50	+21.7	+14.6	−6.1	+20	+54.5	+250	∞	∞	∞	+38
1928–1929: Precipitation:													
Mount Wilson	0.95	2.10	3.30	3.42	3.59	2.94	3.84	0	0.31	0	0	0.23	20.68
Santa Anita Ranger Station	0.80	2.19	4.87	2.75	2.97	3.81	4.92	0	0.36	0	0	0.47	23.14
Run-off:													
Santa Anita Creek	0.02	0.05	0.17	0.17	0.30	0.43	0.74	0.18	0.08	0.02	0.01	0.01	2.17
Fish Creek:													
Actual	0.05	0.08	0.32	0.30	0.51	0.73	0.74	0.18	0.08	0.01	0	0.01	3.01
Normal	0	0.04	0.19	0.19	0.32	0.50	0.89	0.15	0.04	0	0	0	2.32
Increase†, in inches	+0.05	+0.04	+0.13	+0.11	+0.19	+0.23	−0.15	+0.03	+0.04	+0.01	0	+0.01	+0.69
Increase, percentage	∞	+100	+68.5	+57.9	+59.4	+46.0	−16.8	+20	+100	∞	0	∞	+29.7
1929–1930: Precipitation:													
Mount Wilson	0.07	0	0	8.39	1.22	8.02	1.29	4.01	0	0	0	0	23.00
Santa Anita Ranger Station	0.16	0	0	9.47	1.25	6.90	0.93	3.97	0	0	0	0	22.68
Run-off:													
Santa Anita Creek	0.01	0.02	0.03	0.33	0.14	0.78	0.26	0.47	0.15	0.04	0.02	0.01	2.26
Fish Creek:													
Actual	0.02	0.03	0.04	0.65	0.13	1.24	0.25	0.53	0.15	0.03	0.01	0.01	3.09
Normal	0	0	0.01	0.43	0.13	0.91	0.26	0.50	0.10	0.01	0	0	2.35
Increase†, in inches	+0.02	+0.03	+0.03	+0.22	−0	+0.33	−0.01	+0.03	+0.05	+0.02	+0.01	+0.01	+0.74
Increase, percentage	∞	∞	+300	+51.2	−0	+36.3	−3.8	+6.0	+50	+200	∞	∞	+31.5

† Negative sign denotes decrease.
‡ Incomplete.

excess. In Fig. 6 an average annual increase of 1.55 in., or 28.7%, is indicated from Fish Creek during the first six years after complete denudation by the forest fire. On this diagram the dashed line from 1925 to 1930 represents the normal excess.

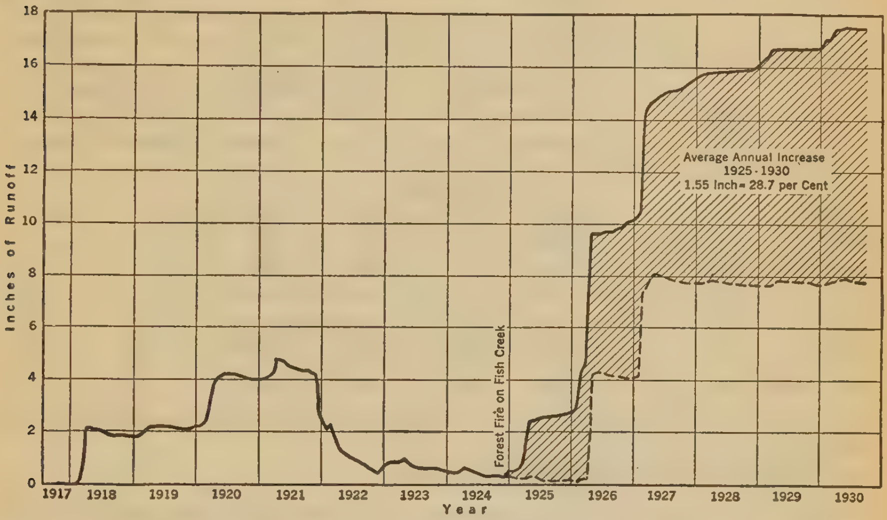

FIG. 6.—ACCUMULATED EXCESS OF RUN-OFF FOR FISH CREEK OVER SANTA ANITA CREEK IN SOUTHERN CALIFORNIA

Distribution of Increase in Annual Run-Off.—The opinion most commonly accepted is that the entire increase of run-off that follows the removal of forest cover occurs during flood periods, and that during low-water periods the run-off is decreased. It was apparent, therefore, that the increase during flood periods and also during the remainder of the year should be studied.

The floods occurring in the two areas studied are very different in type, those in the Wagonwheel Gap area being caused by melting snow as a result of high temperatures and occasional warm light rains, and those in Southern California almost entirely from rain. Stream flow in the Wagonwheel Gap area is on the increase from early April to a peak about May 20, followed by a gradual decline through the remainder of May, all of June, and the first part of July.

Table 4 shows the dates of disappearance of snow from Areas *A* and *B* during two years before and two years after deforestation; the average date of snow disappearance; and the dates of the maximum daily discharge. "Date of snow disappearance" is defined by the authors of *Weather Bureau Review, Supplement No. 30,* as "the first regular observation data on which a trace or no snow was recorded, provided that at no subsequent date a depth of 3 in. or more was recorded." Between fifteen and twenty snow scales were located in each area.

These dates are presented to show the lag between the disappearance of the snow and the peak discharge in the stream. These data and the reasonable assumption that there was no sheet run-off after the snow had gone are the basis for the statement made subsequently under the heading, "Maximum Daily Discharge," that "the gain is due to sub-surface flow as practically all the snow had melted prior to the date of maximum peak."

TABLE 4.—Relation Between Disappearance of Snow and Maximum Daily Discharge in Wagonwheel Gap Area

Year	Date Snow Disappeared		Date of Maximum Daily Discharge		Number of Days Between	
	Area *A*	Area *B*	Area *A*	Area *B*	Area *A*	Area *B*
First Period						
1913 *	April 19	April 22	May 1	May 12	12	20
1917 *	May 25	May 29	June 4	June 6	10	7
Eight years' average †	May 3	May 5	May 14	May 20	11	15
Second Period						
1920 *	May 10	May 5	May 23	May 24	13	9
1925 *	April 23	April 20	May 11	May 15	18	19
Seven years' average ‡	May 8	May 6	May 15	May 17	7	11

* Average dates of melting are published in *Weather Bureau Review, Supplement No. 30*, for only these 4 years.
† Average for the 8 years covered by the first period.
‡ Average for the 7 years covered by the second period.

Only once during the fifteen years of record in the Wagonwheel Gap area was there a flood not attributable mainly to melted snow, and this occurred during October, 1911, prior to deforestation. On the other hand, the floods in the Southern California area may occur at any time after rains, although the most usual time is between December and May. A flood in the Southern California area comes to a peak within a few hours after the rain and seldom lasts more than three days.

In this study the term, "flood period," is roughly defined as a period of major surface run-off as distinguished from the period during which the streams are fed mainly, if not entirely, from sub-surface sources. Owing to the nature of the run-off in the California area the flood periods there are well defined and usually extend over a period of a few days. The run-off during these periods is usually greater than 4 sec-ft. per sq. mile. In the Wagonwheel Gap area the flood period is difficult of determination, but that area has been assumed to be in flood when the daily run-off exceeds 0.05 in. over the drainage basin, or 1.35 sec-ft. per sq. mile, a rate a little less than one-half the average maximum daily run-off. On the average, the period of flood run-off thus determined embraces about 25 days, generally from May 10 to June 5, or somewhat less than 10% of the time.

Table 5 shows, for both areas, the distribution of the difference between the run-off in the flood periods as defined and that for the remainder of the year. In the Wagonwheel Gap area the maximum increase occurred during the third year after deforestation. There was a decrease during the flood period in the first and last years, and the principal increases occurred during the second, third, fourth, and fifth years. The increase during the non-flood period lessened annually from the third year to the last, as would be expected.

In the Southern California area the maximum increase occurred during the second year after the fire. The increase during the flood period occurred mainly in the first three years and was practically negligible in the last three years. The increase during the non-flood period was annually less from the second year to the last.

TABLE 5.—DISTRIBUTION OF INCREASE OR DECREASE, IN INCHES, IN RUN-OFF DURING THE SECOND PERIOD

Year ending September 30	Total increase	Increase or decrease during flood periods	Increase during remainder of year	Year ending September 30	Total increase	Increase or decrease during flood periods	Increase during remainder of year
WAGONWHEEL GAP AREA				SOUTHERN CALIFORNIA AREA			
1920	0.297	—0.039	0.336	1925	2.47	+1.51	0.96
1921	1.529	+0.698	0.831	1926	3.12	+1.61	1.51
1922	2.141	+1.212	0.929	1927	1.58	+0.85	0.73
1923	1.195	+0.600	0.595	1928	0.68	+0.08	0.60
1924	1.204	+0.760	0.444	1929	0.69	+0.15	0.54
1925	0.310	+ 000	0.310	1930	0.74	+0.23	0.51
1926	0.019	—0.008	0.027	....	....	....	....

The data of Table 5 show conclusively that the increase in run-off after the deforestation and fire was not confined entirely to flood periods.

Maximum Daily Discharge and Date of Occurrence.—Table 6 shows that the maximum daily discharge, in second-feet per square mile, under natural

TABLE 6.—MAXIMUM DAILY DISCHARGE, IN SECOND-FEET PER SQUARE MILE

FIRST PERIOD						SECOND PERIOD					
Year	AREA *A*		AREA *B*		Ratio, $\frac{B}{A}$	Year	AREA *A*		AREA *B*		Ratio, $\frac{B}{A}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
WAGONWHEEL GAP AREA											
1912	May 20	4.73	May 22	6.28	1.33	1920	May 23	6.10	May 24	8.64	1.42
1913	May 1	0.97	May 12	1.08	1.11	1921	May 13	2.16	May 15	3.95	1.83
1914	May 11	1.95	May 16	2.31	1.19	1922	May 21	3.22	May 22	5.98	1.86
1915	May 19	2.16	May 20	2.17	1.01	1923	May 20	2.36	May 21	4.28	1.82
1916	May 11	2.61	May 14	3.03	1.16	1924	May 11	2.96	May 15	5.04	1.70
1917	June 4	4.40	June 6	5.40	1.23	1925	May 11	0.74	May 11	1.08	1.46
1918	May 6	0.45	May 21	0.35	0.78	1926	May 8	1.12	May 8	1.90	1.70
1919	May 6	2.57	May 17	3.03	1.18	....	....	.....	....	.....	.....
Average	....	2.48	....	2.95	1.19	Average	....	2.67	....	4.41	1.65
SOUTHERN CALIFORNIA AREA											
Year	SANTA ANITA CREEK		FISH CREEK		Ratio, $\frac{F}{SA}$	Year	SANTA ANITA CREEK		FISH CREEK		Ratio $\frac{F}{SA}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
1918	Mar. 11	14.7	Mar. 11	29.7	2.04	1925	Mar. 6	0.10	Mar. 6	4.77	47.7
1919	Feb. 11	1.62	Feb. 11	1.54	0.95		Mar. 29	0.14	Mar. 29	4.92	35.2
1920	Mar. 2	6.8	Mar. 2	10.8	1.59		April 4	2.38	April 4	20.60	8.65
	Mar. 22	5.7	Mar. 22	12.8	2.25		April 22	0.30	April 22	4.62	15.4
1921	Mar. 13	10.4	Mar. 13	18.5	1.78	1926	Feb. 13	4.38	Feb. 13	13.7	3.13
1922	Feb. 9	33.3	Feb. 9	44.6	1.34		April 7	29.8	April 7	63.0	2.11
1922	Dec. 13	8.57	Dec. 13	9.85	1.15	1927	Feb. 16	41.0	Feb. 16	74.2	1.81
1924	Mar. 26	1.52	Mar. 26	2.16	1.42	1927	Dec. 10	0.40	Dec. 10	1.62	4.05
						1929	Mar. 10	2.67	Mar. 10	6.31	2.36
Average	....	8.35	....	12.98	1.55	1930	Jan. 15	1.90	Jan. 15	6.46	3.40
							Mar. 15	2.38	Mar. 15	6.00	2.52

conditions from the Southern California area is more than five times greater than the maximum daily discharge from the Wagonwheel Gap area. The average ratio of the discharge of Area *B* to that of Area *A* under natural conditions was 1.19, and under deforested conditions, 1.65, an increase of 39 per cent. The average ratio of the discharge of Fish Creek to that of Santa Anita Creek under natural conditions was 1.55. During the first two years after the fire the maximum ratio was as high as 47.7. For the first four storms occurring in the first year after the fire the average increase was about 1 700%, based on a comparison of ratios. By the second year after the fire the ratios had dropped to practically normal.

In the Wagonwheel Gap area the effect of the removal of the trees was to advance the date of the maximum daily run-off of Area *B* about three days. In the Southern California area, the fire caused no appreciable change in the occurrence of maximum daily discharge.

Fig. 7 shows the actual and computed normal run-off, in second-feet per square mile, of Area *B* and Fish Creek during the typical high-water period occurring three years after deforestation and denudation (April, May, and June, 1922, on Area *B*, and February, March, and April, 1927, on Fish Creek). The shaded area represents the increase or decrease of surface run-off. For clearness an insert is shown with an exaggerated time scale for a few days during the flood peak in the Fish Creek area (Fig. 7(*c*)). It will be observed that practically all the increase on both areas occurred on the rising stage. During the falling stages decreases were common.

Maximum Peak Run-Off.—In the foregoing remarks no reference has been made to peak discharges. In the Wagonwheel Gap area maximum peak discharge results from melting snow, and the ratio between maximum peak and maximum daily discharge is slightly more than 1 : 1. In a semi-arid country, such as the Fish Creek Basin, the maximum discharge depends almost entirely on the rainfall. Immediately after the fire the bare canyon walls were much more conducive to run-off than the plant-covered surface that

TABLE 7.—MAXIMUM PEAK DISCHARGE AND MEAN DAILY DISCHARGE, IN SECOND-FEET PER SQUARE MILE, IN THE FISH CREEK AREA, SOUTHERN CALIFORNIA

Year	Date	Peak discharge	Daily discharge	Ratio, peak to daily discharge	Year	Date	Peak discharge	Daily discharge	Ratio, peak to daily discharge
FIRST PERIOD					SECOND PERIOD				
1917–18...	March 10	50.8	14.7	3.5	1924–25..	April 4	335	20.6	16.2
1918–19...	February 11	3.2	1.5	2.1	1925–26..	April 7	292	63.0	4.6
1919–20...	March 2	39.2	10.8	3.6	1926–27..	February 16	145	74.2	2.0
1920–21...	March 13	44.0	18.5	2.4	1927–28..	February 4	14.9	4.6	3.2
1921–22...	February 9	77.7	44.6	1.7	1928–29..	March 10	10.9	6.3	1.7
1922–23...	December 12	28.6	9.8	2.9	1929–30..	January 15	11.1	6.5	1.7
1923–24...	March 26	8.9	2.2	4.0	....	....	.....	.....	....
Average..	....	36.0	14.6	2.5	....	....	.....	.....	....

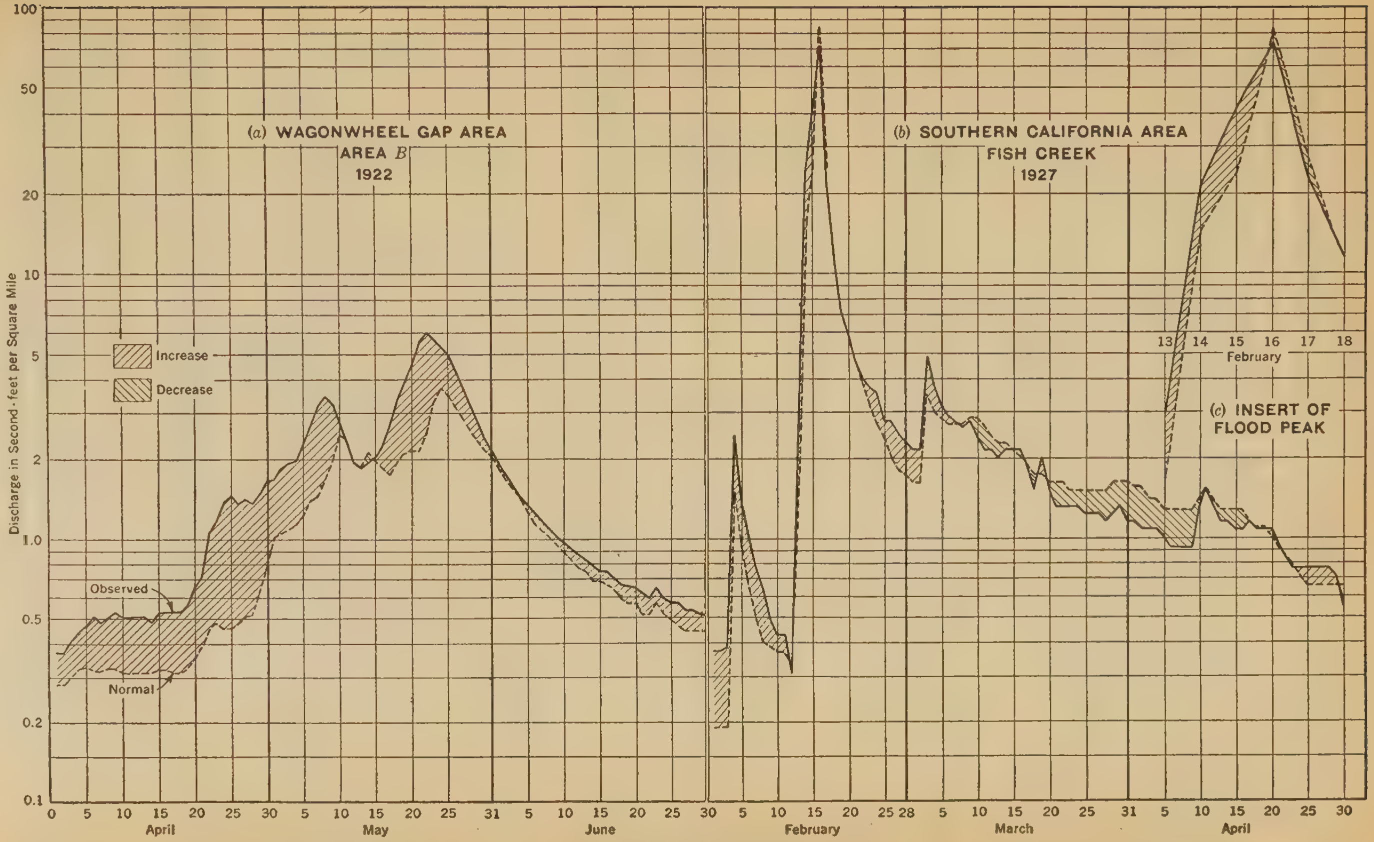

FIG. 7.—OBSERVED AND NORMAL RUN-OFF OF AREA *B* AND FISH CREEK DURING THE STORM PERIOD FOR THE THIRD YEAR AFTER CHANGE

existed prior to the fire. The greatest effect was felt during the first year after denudation. This is well expressed in Table 7, which shows the maximum peak discharges for each year, together with the mean daily discharge for the same day.

During the first period the average peak discharge was 2.5 times the mean daily discharge. In the second period, the first year (1924-25) showed a ratio considerably greater than would have been expected under normal conditions. In the second year, the ratio of the peak to the daily discharge was greatly reduced, but still was larger than any of the ratios of the first period.

Summer Run-Off.—Table 8 shows the run-off for July to October for both areas for both periods. In the Wagonwheel Gap area the ratio of the run-off of Area *B* to that of Area *A* for the first period was 0.93 and for the second period, 1.03. In the Southern California area the ratio of the run-off of Fish Creek to that of Santa Anita Creek for the first period was 0.45 and for the second period, 1.69.

TABLE 8.—SUMMER RUN-OFF (JULY TO OCTOBER), IN INCHES

FIRST PERIOD				SECOND PERIOD			
Year	Area *A*	Area *B*	Ratio, $\frac{B}{A}$	Year	Area *A*	Area *B*	Ratio, $\frac{B}{A}$
			WAGONWHEEL GAP AREA				
1912	1.641	1.596	0.97	1920	1.481	1.485	1.00
1913	1.168	1.237	1.06	1921	1.602	1.651	1.03
1914	1.465	1.262	0.86	1922	1.457	1.474	1.01
1915	1.223	1.123	0.92	1923	1.526	1.549	1.02
1916	1.507	1.394	0.90	1924	1.325	1.421	1.07
1917	1.756	1.499	0.85	1925	1.108	1.239	1.12
1918	0.915	0.965	1.05	1926	*0.824	*0.802	0.97
1919	1.320	1.206	0.91		....	....	....
Average	1.375	1.285	0.93	Average	1.333	1.375	1.03
			SOUTHERN CALIFORNIA AREA				
Year	Santa Anita Creek	Fish Creek	Ratio, $\frac{F}{SA}$	Year	Santa Anita Creek	Fish Creek	Ratio, $\frac{F}{SA}$
1918	0.30	0.13	0.43	1925	0.06	0.16	2.67
1919	0.11	0.05	0.45	1926	0.21	0.38	1.81
1920	0.17	0.05	0.29	1927	0.33	0.60	1.82
1921	0.38	0.11	0.29	1928	0.06	0.10	1.67
1922	1.01	0.57	0.56	1929	0.05	0.04	0.80
1923	0.27	0.09	0.33	1930	*0.07	*0.05	0.71
1924	0.06	0.01	0.17		....	....	....
Average	0.38	0.17	0.45	Average	0.13	0.22	1.69

* October record not available.

Table 9 shows the increase in run-off for July to October in the second period for both areas. The average increase in Area *B* during the seven years after deforestation was 12%, and the average increase in the Fish Creek area for the six years after the fire was 475 per cent.

TABLE 9.—INCREASE IN SUMMER RUN-OFF, JULY TO OCTOBER, IN INCHES AND PERCENTAGE, DURING THE SECOND PERIOD

WAGONWHEEL GAP AREA				SOUTHERN CALIFORNIA AREA			
Year	Normal discharge	INCREASE		Year	Normal discharge	INCREASE	
		Inches	Percentage			Inches	Percentage
1920	1.340	0.145	11	1925	0.01	0.15	1 500
1921	1.438	0.213	15	1926	0.05	0.33	660
1922	1.323	0.151	11	1927	0.14	0.46	304
1923	1.403	0.146	10	1928	0	0.10	∞
1924	1.226	0.195	16	1929	0	0.04	∞
1925	1.127	0.112	10	1930	*0.01	*0.04	*400
1926	*0.856	*0.074	*9		....	....	....
Average	1.244	0.148	12	Average	0.04	+0.19	475

* October not available.

Fig. 8 shows the actual and the computed normal run-off during August and September, the period of low summer run-off, for the third year after the change (1922 for Area *B* and 1927 on Fish Creek). The cross-hatched area represents the typical increase in run-off during the summer.

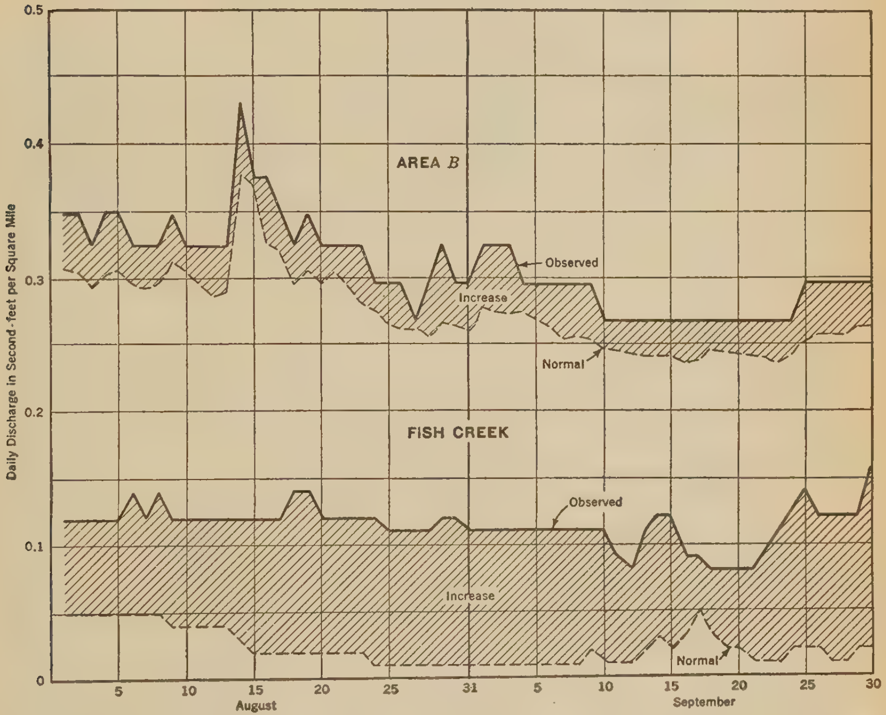

FIG. 8.—OBSERVED AND NORMAL SUMMER RUN-OFF OF AREA *B* AND FISH CREEK FOR THE THIRD YEAR AFTER THE CHANGE

Fig. 9 shows graphically the increase in summer for both areas for the entire period. This diagram shows the accumulated differences in run-off for the two areas for the first period and the accumulated differences between the observed run-off and the normal run-off of both areas for the second period.

Minimum Daily Discharge and Date of Occurrence.—Table 10 shows the minimum daily discharge during the summer for both areas for both periods. Under normal conditions, the minimum rate of discharge in the Wagonwheel Gap area is between seven and eight times greater than that in the Southern California area. The ratio between the minima in Area *B* and Area *A* under natural conditions was 0.98 during the first period; after deforestation, the ratio was 1.10, indicating an increase in run-off of about 12 per cent. Fish Creek was dry at times in four out of the seven years preceding the fire. The ratio between the minima on Fish Creek and Santa Anita Creek based on the seven years was 0.32.

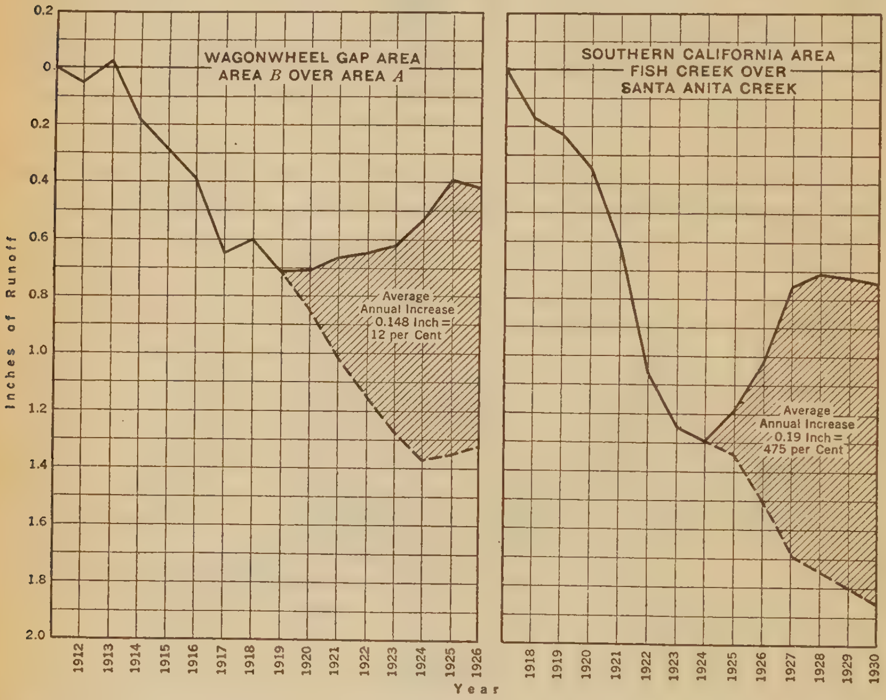

FIG. 9.—ACCUMULATED EXCESS OF RUN-OFF OF AREA *B* OVER AREA *A* AND OF FISH CREEK OVER SANTA ANITA CREEK DURING THE SUMMER PERIOD

After the removal of the forest cover in Area *B*, the date of occurrence of the summer minimum was delayed about five days. After the fire in the Fish Creek area the date of occurrence of the summer minimum was delayed more than a month.

Table 11 shows the minimum daily discharge during the winter for the Wagonwheel Gap area. The average ratio of the minimum in Area *B* to that in Area *A* during the first period was 1.30, or 1.24 if 1915, when the winter minimum for Area *A* was abnormal, is excluded. The ratio during the second period was 1.27 indicating little if any change in winter minima. Inasmuch as the date of occurrence and to a large extent the magnitude of winter minima are a direct result of low temperatures no material change was to be expected. In the Southern California area winter temperatures are not sufficiently low to cause a period of corresponding low run-off.

TABLE 10.—MINIMUM DAILY DISCHARGE DURING SUMMER, IN SECOND-FEET PER SQUARE MILE

First Period						Second Period					
Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$	Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$
	Date	Dis-charge	Date	Dis-charge			Date	Dis-charge	Date	Dis-charge	
					Wagonwheel Gap Area						
1912	September 16	0.283	September 7	0.279	0.99	1920	August 30	0.253	August 31	0 265	1.05
1913	August 31	0.205	August 8	0.221	1.08	1921	September 14	0.276	September 14	0 312	1.13
1914	September 8	0.247	September 7	0.224	0.91	1922	September 17	0.247	September 14	0 265	1.08
1915	September 12	0.183	September 12	0.183	1 00	1923	September 11	0.256	September 11	0.298	1.16
1916	August 28	0 259	August 28	0.237	0 92	1924	August 31	0 230	August 22	0 250	1.09
1917	September 6	0.268	September 4	0.236	0.88	1925	August 17	0.205	August 17	0.232	1 13
1918	August 25	0.159	August 1	0.171	1.08	1926	September 10	0.173	September 1	0.179	1.03
1919	September 4	0.207	September 4	0.225	1.09	....	....	.....	....	.....	....
Average	....	0.226	....	0.222	0.98	Average	....	0.234	....	0.257	1.10

					Southern California Area						
Year	Santa Anita Creek		Fish Creek		Ratio, $\frac{B}{A}$	Year	Santa Anita Creek		Fish Creek		Ratio, $\frac{B}{A}$
	Date	Dis-charge	Date	Dis-charge			Date	Dis-charge	Date	Dis-charge	
1918	September 11	0 03	July 11	0.02	0.67	1925	July 19	0.01	August 18	0 02	2.00
1919	July 31	0.01	June 26	0	∞	1926	August 25	0.02	September 20	0 05	2.50
1920	August 9	0 02	July 29	0	∞	1927	August 25	0.04	September 12	0.08	2.00
1921	September 6	0 02	August 27	0	∞	1928	July 23	0.01	August 20	0	∞
1922	September 25	0.11	September 25	0.03	0.27	1929	August 18	0	August 4	0	∞
1923	September 2	0 02	July 11	0.02	1.00	1930	August 22	0.01	July 27	0	∞
1924	July 2	0.01	June 15	0	∞	....	....	.....	....	.....	....
Average	....	0.031	....	0.010	0.32	Average	....	0.015	....	0.025	1.67

Erosion: Wagonwheel Gap Area.—In connection with the studies of run-off, quantitative measurements were made, by means of settling basins, of the silt carried by the streams of the two areas near Wagonwheel Gap. During the first eight years the average annual quantity of silt deposited in the collection basin was 691.5 lb. for Area *A* and 568.5 lb. for Area *B*, or 3.15 and 2.85 lb., respectively, per acre. During the seven years after deforestation the average quantities collected were 477 lb. from Area *A* and 3 340 lb. from Area *B*, or 2.15 and 16.7 lb., respectively, per acre, showing about an eightfold increase in the ratio of Area *B*. The higher value represents only a few shovelfuls per acre per year, and, as Messrs. Bates and Henry state,[9]

> "It is only fair to the present discussion to point out that even this large quantity does not represent erosion in the commonly accepted sense of a destructive process."

As to the visible effect, they say:

> "The erosion from the slopes was practically invisible except for one small gully formed from a skid trail, and even the erosion from this appeared to be largely deposited upon a leveled road, without reaching the stream channel."

TABLE 11.—MINIMUM DAILY DISCHARGE DURING WINTER, IN SECOND-FEET PER SQUARE MILE, IN THE WAGONWHEEL GAP AREA

Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$	Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
	First Period						Second Period				
1912	Feb. 28	0.234	Feb. 28	0.279	1.19	1920	Mar. 8	0.195	Feb. 15	0.222	1.14
1913	Mar. 1	0.210	Mar. 16	0.266	1.27	1921	Feb. 10	0.220	Feb. 10	0.278	1.26
1914	Feb. 14	0.203	Feb. 16	0.233	1.15	1922	Jan. 27	0.170	Jan. 23	0.289	1.70
1915	Dec. 16	0.109	Dec. 17	0.216	1.98	1923	Mar. 18	0.225	Mar. 15	0.273	1.21
1916	Feb. 6	0.182	Feb. 1	0.227	1.25	1924	Feb. 5	0.221	Feb. 6	0.283	1.28
1917	Dec. 11	0.188	Dec. 15	0.266	1.41	1925	Feb. 23	0.231	Feb. 26	0.273	1.18
1918	Feb. 28	0.178	Feb. 3	0.248	1.39	1926	Feb. 27	0.191	Feb. 6	0.214	1.12
1919	Feb. 17	0.180	Feb. 1	0.195	1.08	......	....	.....	....	.....	....
Average: Including 1915......	....	0.185	....	0.241	1.30	Average.	....	0.207	....	0.262	1.27
Omitting 1915......	....	0.197	....	0.245	1.24						

Erosion: Southern California Area.—On Fish Creek the rapid run-off during the first year after the fire (1924-25) caused considerable erosion to the side walls of the canyon. Beginning December 16, 1924, a series of silt samples were taken in connection with the current meter measurements made at irregular intervals by the U. S. Geological Survey at each of the gauging stations in the burned area. Most of these samples were collected at times when the discharge in the streams was at storm stage. For this reason the figures obtained are considerably above the average for the entire year.

The first rains after the fire washed the ash and loose material from the side walls of the canyon into the stream, to such an extent that by December 16, 1924 (three months after the fire), the entire stream bed had been buried. From a series of pools and rocky rapids, it had become a channel of more or less uniform cross-section with a fairly uniform gradient, sufficient to cause a considerable increase in the velocity. Three current meter measurements made on April 4, 1925, at discharges of 105 to 74 sec-ft., showed mean velocities of 10.5 to 9 ft. per sec. Under normal conditions this discharge would not have had a velocity of more than 2 ft. per sec. As late as June 4, 1925, when the discharge was 3.1 sec-ft., the mean velocity was 4.8 ft. per sec. To this uniform gradient of the stream bed with the increased velocity is partly due the rapid run-off and the unusually large peak storm discharge.

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[9] *Monthly Weather Review, Supplement 30.*

The silt samples were collected in quart jars. As a rule the mouth of the jar was held half way between the bottom and the surface of the stream. Owing to the shallowness of the stream and the greatly increased velocity, part of the material collected in the samples consisted of the heavier particles which, ordinarily, would form the rolling load, but were then temporarily in suspension.

TABLE 12.—SILT IN SAMPLES FROM SOUTHERN CALIFORNIA AREA

Location	PERCENTAGE BY VOLUME			PERCENTAGE BY WEIGHT		
	Sand	Ash	Total	Sand	Ash	Total
Rogers Creek:						
December 16, 1924, 11:30 A. M.	25	42	67	25	14	39
December 16, 1924, 3:30 P. M.	18	38	56	18	13	31
March 6, 1925	25	25	50	32	6	38
March 29, 1925, 10:25 A. M.	28	16	44	28	6	34
April 4, 1925, 9:55 A. M.	24	24	48	25	10	35
April 20, 1925, 9:20 P. M.	10	12	22	......	......	9.8
April 7, 1926, 5:30 P. M.	......	......	5.5	......	......	3.8
Fish Creek:						
December 16, 1924, 12:20 P. M.	......	......	20	......	......	5.9
March 31, 1925	......	......	20	......	......	14.3
Sawpit Creek:						
December 16, 1924, 2:00 P. M.	10	40	50	12	14	26
April 4, 1925	30	36	66	30	10	40

After the samples had been allowed to settle for three years the results shown in Table 12 were obtained. As a rule, it was possible to separate the ash from the sand, because the sand settled first with the ash on top of it. In most of the samples the division between the sand and ash was well marked. The material called sand in Table 12 consisted mainly of small irregular particles washed off the canyon walls. Included with the ash was some unburned organic matter which usually floated on the surface.

Under normal conditions the erosion in each of the three areas represented in Table 12 is negligible. Prior to the fire there was practically no movement of silt on Fish Creek except during the storm run-off, and by the third or fourth day after a storm the water was practically clear. In the first year after the fire the large deposit of silt from the burned-over area caused considerable damage to orchards, railroads, and highways adjacent to the mountains. In the second year the quantity of silt moved was much smaller and caused little if any damage. By 1930, the silt had been reduced to almost normal, and the appearance of the stream bed was much the same as before the fire.

CONCLUSIONS

The conclusions to be drawn from this paper may be discussed under seven heads, as follows:

1.—Total Run-Off.—Forests did not "conserve the water supply," because after their removal there was an increase in average annual yield amounting to 15% in a mountain area in Colorado and 29% in a Southern California coastal mountain area.

2.—Distribution of Increase in Run-Off.—Contrary to the widely quoted opinion the increase in run-off is not confined wholly to flood periods. In both the Wagonwheel Gap area and in the Southern California area, 52% of the increase occurred during the non-flood period. The flow during the non-flood period is derived from sub-surface storage. The increase during non-flood periods results from either (*a*) increased sub-surface flow and storage; or (*b*) decreased transpiration; or (*c*) a combination of both. More of the precipitation may enter storage after removal of vegetative covering for the following reasons: (1) Less interception by trees, undergrowth, tree litter, and humus; and (2) faster melting of snow with corresponding decrease in evaporation. The gradual lessening of the increase during the non-flood period in both areas after the second or third year may be a reflection of gradual increase in plant growth and corresponding increase in transpiration.

3.—Maximum Daily Discharge.—In the Wagonwheel Gap area there was an average increase of 46% in maximum daily discharge after deforestation. This gain is due to increased sub-surface flow, as practically all the snow had melted several days prior to the date of maximum peak. During flood periods following the snow-melting the discharge in Area *B* uniformly reached a peak three days or more after that in Area *A*, but after the storm of October 5, 1911, the peak in both areas occurred at the same time. This was the only so-called flood that resulted from rainfall and indicates that surface run-off from both areas reached the gauging station at practically the same time. In the Southern California area the four storms occurring during the first year after the fire resulted in an increase of 1 700% in the maximum daily discharge. The peak discharge, which was ordinarily 2.5 times the maximum daily discharge prior to the fire, increased to 16.2 times on April 4, 1925, the maximum peak for the period of record. The floods in the Southern California area usually result from rainfall and represent direct surface run-off.

The removal of vegetative covering clearly increases normal flood heights. Beginning in the second year after the fire on Fish Creek, the flood peak discharges were practically the same as those which occurred before the fire (see Table 7), indicating that the new growth, small as it was, exercised practically the same effect as the original cover in reducing flood crests.

The gradual increase in vegetative covering on the Wagonwheel Gap Area *B* after deforestation had little effect on the increase in flood run-off. (See Table 6.) This was to be expected in that most of the flood run-off in this area reaches the stream through underground channels.

The earlier melting of the snow in the Wagonwheel Gap area resulted in an advance of three days in the flood peak. Where rain passes directly into streams without entering the ground the flood peak for any small element of drainage area occurs so soon after the storm that removal of vegetative cover has little effect on the time element. In the Southern California area the peak was advanced only a few hours.

4.—Summer Run-Off.—It is almost universally believed that forests or vegetative covering will increase summer run-off and shorten the low-water period through the exercise of storage functions. This belief is an outstanding fallacy in so far as these two widely different areas are concerned.

The summer flow (July to October) in the Wagonwheel Gap area showed an average annual increase of 12% during the seven years after deforestation, and the Southern California area, an average annual increase of 475% during the six years after the fire. The absolute average increase of run-off, in inches, for both areas was practically the same. (See Table 7.) The high percentage of increase shown for Fish Creek results from the extremely small summer run-off of that stream under normal conditions. This increase is probably a result of increase in sub-surface storage and decrease in transpiration.

5.—*Minimum Daily Discharge and Date of Occurrence.*—Coincident with the increase in summer run-off there was an increase in the average summer minimum and the period of low-water run-off was considerably shortened. In the Wagonwheel Gap area the average minimum was increased about 12% and the time of occurrence delayed about 5 days. (See Table 10.) In the Southern California area the average minimum was increased more than 400% and the time of occurrence was delayed about 30 days.

6.—*Winter Minimum.*—Deforestation made no appreciable change in the low flows which occurred during the winter in the Wagonwheel Gap area.

7.—*Erosion.*—Erosion results from surface flow. In the Wagonwheel Gap area there was practically no evidence of erosion after deforestation, and this was to be expected because there was little direct surface run-off either before or after deforestation. In the Southern California area complete denudation increased erosion as a direct result of the increased surface run-off. So far as the surface features of the Fish Creek Basin were concerned the erosion did not destroy any present use. However, deposition of eroded material and ash carried by the streams the first year after the fire was materially injurious to agricultural lands and transportation rights of way below the canyon. As it is clearly shown in Tables 6 and 7 that the growth of new vegetative covering by the second year after the fire was nearly as effective in reducing normal flood run-off as the original cover, it may well be argued that if the Fish Creek area had been deforested by cutting of the vegetative cover, as in the Wagonwheel Gap area, without the destruction of the roots and tree litter, the erosion would have been practically the same as with the original vegetative covering.

DEDUCTIONS

If the facts as found are generally applicable to other similar areas which in their natural state support forest growth, then the hydraulic engineer and the water user must weigh such protective influence as forest cover may have in lowering normal flood crests and in preventing or retarding erosion against the detriment resulting from deceased yield and reduced low-water flow during the critical growing period. If the small growth that springs up immediately after deforestation or denudation exercises practically the same effect as forests in reducing normal flood crests and in preventing erosion—without the detrimental effect which forest cover is shown to have on annual flow and flow during the summer low-water periods—then in basins where shortages in water supply are becoming critical or where abnormal expenditures have been made to augment water supplies, the maintenance of forests or reforestation for the "conservation of water supply" may have an effect exactly opposite to that desired.

The writers are lovers of forests and have a keen appreciation of the value of water in the economic development of the United States. The value of the forests as a source of wood for building material and other industrial purposes and as playgrounds is alone sufficient to warrant their preservation and maintenance. Scientifically determined facts apparently do not warrant their development solely for the conservation of water supply. In regions where such supply is a controlling economic factor, careful study is needed to determine whether the value of increased water supply and better-sustained minimum flow which are shown to obtain without forests, does not outweigh the benefits of lowered normal flood flows and decreased erosion produced by forests, especially if these benefits can be obtained by shrubs or other small growth without the loss of water occasioned by forest growth.

DISCUSSION

C. G. Bates,[10] Esq. (by letter).—Since the publication in 1928 of the report[11] of the Wagonwheel Gap stream-flow experiment, the writer has lived in momentary expectation of just such inferences as Messrs. Hoyt and Troxell have made from it. Boldly stated, the authors' conclusion, based on records from two pairs of Western water-sheds, is that vegetation on a water-shed is a detriment to the yield of water. Their revolutionary scheme, brought to its logical conclusion, is to obtain a higher yield of water by means of the simple expedient of removing the vegetation from the water-shed, and with that, eventually, the soil as well. The tone of the paper makes it appear to be directed against foresters who see in the forest cover a beneficial effect.

The writer does not propose to quarrel or quibble over the facts from the water-shed under discussion. By using a different method of analyzing the data obtained from the Wagonwheel Gap experiment, the authors have changed the run-off percentages by a few tenths of 1 per cent. The stream-flow records in Southern California were obtained without adequate precipitation records for the two water-sheds. These are minor points and are immaterial to the main discussion. On the whole, the authors have stated the facts with careful veracity.

However, they did not need to go to all the trouble of trying to prove that forest cover—or for that matter any vegetative cover—is a source of loss of water to the water-shed. Foresters were the first to demonstrate that forests use water in large quantities, possibly in greater quantities than any other form of vegetation, except deep-rooted perennial crops, such as alfalfa. Foresters, again and again, have pointed out the probability that a water-shed, bare of vegetation, is capable of yielding a larger total quantity of water than the same water-shed if covered with a forest, and the denser and more luxuriant the vegetation the greater the loss of water to the water-shed. This is a biological phenomenon no longer in need of proof.

What foresters have claimed and what universal observation since time immemorial has proved is that a forest cover under certain conditions tends to equalize the run-off throughout the year, to increase the storage capacity of the water-shed (especially where the soils are thin or heavy and impervious), and to reduce soil erosion. Foresters have further shown that the beneficial effect of the forest cover varies with the climate, the character of precipitation, and, above all, the character of the soil.

Criticism of this paper is not so much against the authors' conclusions, as they relate to the specific Western water-sheds, but against their attempt to generalize from these water-sheds—which are extreme in character of soil and climate—as to the protective value of the forest under all conditions.

The authors have omitted many facts that are vital to any logical and sane discussion of the subject of forests and stream flow, of "water-shed protection," and of the general utilitarian value of forested *versus* bare lands. While their proposal is carefully guarded and qualified, it tends to leave the impression that sufficient facts have been established to warrant so radical a departure as the elimination of protective forest cover. It is impossible to answer such a proposal in a few words, or even to enumerate more than the most poignant facts.

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[10] Senior Silviculturist, U. S. Forest Service, St. Paul, Minn.

[11] *Monthly Weather Review, Supplement 30.*

The paper lacks essential information on the geology and soils of the two areas, which would permit a student of such matters to rate the tendency toward surface run-off and erosion and the capacities of the water-sheds for deep ground storage by means of which the streams might be maintained through dry weather. It is not, for example, clearly stated that, in the case of the Wagonwheel Gap water-sheds, the coarse crystalline character of the rock and the resultant porosity and thinness of the soil layer practically precluded the possibility of any surface run-off, and, at the same time, distinctly limited the capacity of the water-shed as a whole for storage in the ground, this so-called soil layer having an apparent average depth of only about 6 ft. In the case of the 3-in. rainfall of October, 1911, the impression is given that the peak of flow on both water-sheds was reached during, and by reason of, surface run-off, whereas this peak flow (although augmented by water falling in the streams) represented merely a saturated condition of the entire water-sheds and played only a small part in the entire flow which occurred subsequently. In such a case as this, and in fact in all the stream-flow phenomena of these water-sheds, it was clearly demonstrated that the surface conditions as modified by the existing forest could play but a very small part. This is far from being the case with mountain water-sheds in general.

It is obvious, of course, that the water-sheds at Wagonwheel Gap do not represent quite the extreme of the porous, completely absorbing type of water-shed. The writer happens to be familiar with various sand-hill regions. In Central Nebraska, for example—what with the porous sand as an absorbing mantle, the lack of dissection, the "pocket" conformation (which is characteristic of all sand dunes), and the great depth and storage capacity of the sand reservoir—the streams that drain the region are extremely steady in their flow, and their annual "floods" probably do not exceed the annual low-water discharges by a ratio of more than about 3 to 1. The water-sheds at Wagonwheel Gap had a high-low discharge ratio (daily, not peak discharges) of 12 to 1, and after deforestation Water-Shed *B* showed a ratio of 17 to 1. It is apparent, therefore, that the porous type of water-shed is not likely to show a ratio of more than 25 to 1, over a long period of years.

The omission by the authors of many pertinent facts with regard to the California areas discussed is decidedly confusing. Nothing is said as to the geology and soils of the two water-sheds. From a slight knowledge of Southern California foothill conditions and from the map presented (Fig. 2), the writer assumes that the lower portions of the two water-sheds have relatively deep but impermeable soils, the surface run-off of which contributes largely to the quick discharges after storms, while the upper portions are relatively rocky and barren. It is probably in the latter portion that most of the absorption of the voluminous winter rains occurs, by reason of which the streams (especially the Santa Anita with its second high area on Mt. Wilson) are maintained throughout most, but not all, of the summer months when no rain is recorded. Because of the fact that many of the winter rains in California are gentle and prolonged, there is, however, for the entire water-shed areas greater opportunity for absorption and storage than occurs with quick summer rains or, in many cases, with the melting of accumulated snow. Probably only in their lower reaches, therefore, do these water-sheds represent that completeness of surface run-off which is associated with arid climatic conditions and which is characteristic of the entire foothill fringe of the Western mountainous area and the adjacent plains and deserts.

There are much more extreme conditions than those cited, as in the case of the extremely impervious shaly or clay soils of the Bad Lands of Dakota and Montana. However, even without this extremely impervious type of soil, essentially all stream flow tends to be in the form of surface run-off and the streams "go dry" completely almost every summer and often throughout the greater part of each year. This is quite largely the result of incapacity to form humus and to enrich the surface soil therewith, which, in turn, can probably be ascribed to dry atmospheric conditions more than any other one factor. In short, all organic litter is oxidized so rapidly that there can be no accumulation, except possibly on a fraction of the area on which brush clumps mechanically hold litter and protect it from the sun. There is no chance of developing on such sites a forest canopy in the true sense, although scattered trees generally establish themselves in the less extreme conditions. In the intervening spaces between clumps of brush or trees, the bare mineral soil is generally exposed, these spaces comprising from 30 to 90% of the entire surface. Inevitably, the result is that surface run-off is never entirely precluded and that any effect from the destruction of the vegetation and existing partial humus mantle changes run-off conditions only in degree and not in kind.

The two types of water-shed drawn upon by the authors, therefore, represent almost complete antitheses, the one showing a high absorbing capacity and a relatively steady flow, and low ratio between flood discharge and summer minima, the other being in the "surface run-off" class, exhibiting little ability to utilize whatever deep storage the soil and geology of the areas provide, and frequently "going dry" in the summer, so that its high-low discharge ratio is in many years infinity, and on the average very high.

Between these two classes of water-sheds lies the greater proportion of the forested areas of the United States both in the Western mountains and in the more humid East. Although, of course, depending on the manner in which the surface cover has been treated, the high-low ratios for a large proportion of the water-sheds in the country run from 50:1 to 150:1. Under the more humid conditions which permit the actual development of a normal forest, and which, at the same time (by the more rapid processes of weathering), rarely permit extremely porous soils to exist, an entirely different situation exists. It seems undeniable, if one accepts human experience and observation as worth anything at all, that in one important particular the stream-flow situation is different from that in the two cases cited. This is in the matter of increased, rather than decreased, summer flow as a result of the long-time effects of forest cover and of humus upon the absorptive capacity of the soil.

If, for example, one can picture the difference between 3% run-off from a well-forested area, and 50 to 80% run-off from burned, pastured, and cultivated areas, during rains which pour from 1 in. to 3 in. of water upon the soil in as many hours, one obtains the basis for an entirely different conception of the value of forest cover. This value is obtained, necessarily, by deep storage of the additional water which gets into the ground through a good layer of humus. Much of this storage is undoubtedly beyond the reach of tree roots, so that the surface layer may become completely dried through root absorption, and still the deeper reservoir remains filled and supplies springs throughout long periods of dry weather. It is obvious that this effect on the low-water flow (even admitting a decrease in total annual run-off) arises solely through increased surface absorption and the existence of a potential large reservoir in the deep soil and rock layers. In recent years, and from a multiplicity of sources, the facts regarding the increased absorption due to forest humus have been established beyond a shadow of doubt. The work[12] of Dr. W. C. Lowdermilk, too, has presented an entirely new and much more sane conception of the function of humus in water absorption by soils, so that one no longer thinks of the humus layer as a reservoir in itself, but as supplying the channels through which water may penetrate an otherwise largely impermeable soil.

That all these facts have not been brought together under one comprehensive water-shed experiment, in which the year-long regimen of streams was studied, is unfortunate, but it should scarcely destroy sanity. In fact, the weight of observation on this point is so great that it certainly cannot be set aside because of conflicting observations under conditions of arid climates and limited storage possibilities. The conditions cited by the authors are in no sense "normal," although they may possibly be representative of considerable areas in the West.

The authors, while admitting the occurrence of severe erosion within a few months after the Fish Creek fire (which erosion filled the stream channel with débris and did "considerable damage to orchards, railroads, and highways" below the water-shed), tend to minimize its importance and state in their "Conclusions" that "so far as the surface features of the Fish Creek Basin were concerned the erosion did not destroy any present use." This bland statement of the harmless effect of one fire begs the question and is misleading. Erosion is a time-consuming process—it normally proceeds almost imperceptibly so far as "surface features" are concerned, and destructive erosion gains headway only after the "harmless" type of erosion has proceeded for a considerable time. Messrs. Hoyt and Troxell have shown that the beneficial effects of the fire denudation began to decrease after a couple of years. It would require repeated fires and almost complete elimination of all vegetative cover to maintain such benefits to stream flow. That destructive erosion would soon follow such treatment is a foregone conclusion, and there is only one logical outcome to the use of the methods proposed by the authors for obtaining greater stream flow—to remove the entire soil mantle of the water-sheds and with it every obstacle to quick surface run-off.

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[12] "Influence of Forest Litter on Run-Off, Percolation, and Erosion, *Journal of Forestry*, Vol. XXVIII, No. 4, April, 1930.

It seems scarcely probable that even the people of Southern California would place such stress on increased population and increased water supplies that they would choose to have their entire foothills region assume the barrenness of the upper slopes of Mt. Wilson. Nor is it conceivable that any engineer will argue, in view of the reservoir difficulties which are daily being brought to his attention, that surface run-off carrying a heavy burden of silt is equivalent in practical usefulness to a corresponding volume of clear, subsurface flow. There may be communities in dire need of additional water, but it is not believed there are any that can afford to obtain it by a method so wholly destructive in its design.

As far as the general proposition of "protection forests" on mountainous water-sheds is concerned, progressive erosion as the result of deforestation or complete denudation is the crux of the situation. It is rarely that areas will be found as immune to surface run-off and erosion as those at Wagonwheel Gap, and it may be seriously questioned whether these areas would stand repeated burnings, and (as a logical sequence) grazing of the herbaceous vegetation which sprang up, without showing a great change in respect to these soil-destroying processes. At the other extreme, there is, chronically, some erosion despite the efforts of a feeble vegetative cover to maintain itself, and with the kinds of soils which usually prevail in the foothills areas, unimportant erosion may quickly develop into deep gullying. In areas of the intermediate class, there will be no less a tendency toward soil loss once the forest is removed, except as delayed by the greater depth of humidified soil and the richer vegetation which that soil and the more humid conditions will support. Once this "reserve" has been lost through a conscious effort to keep down the forest, the unprotected soil will give way in much the same manner.

It is for the conservation of the soil resource quite as much as the water resource or timber resource that the forester argues the need of "protection forests." In young mountainous regions, Nature has decreed that the one is indispensable to the other and that the building of soil in the first instance is dependent on the roots of trees and on lower forms of vegetation which grow on rock surfaces; then, on the chemical action of the organic matter which they deposit; and, finally, upon the protection from erosion which is furnished by the complete humus mantle. The forest, in turn, can develop as individual units and, finally, as a solid "stand" only as these soil-forming steps progress and as the storage capacity of the soil increases. Eventually, deep soil deposits are obtained which far surpass in storage capacity the requirements of the protecting forest.

Has the time arrived when Man may assume the audacity to start reversing these great constructive processes of Nature, for the sake of a temporary benefit, to water a few paltry additional acres of crops or to permit a few more people to congregate in the cities? The Engineering Profession is not lacking in audacity, and it frequently defaces and defies Nature for the sake of a temporary gain. When such a proposal as that of Messrs. Hoyt and Troxell is being seriously put forward, it is time for the profession to ask itself whether or not it is truly giving heed to the interests of humanity.

J. E. WILLOUGHBY[13] M. Am. Soc. C. E. (by letter).—This paper is conclusive as evidence that forests do not conserve the water supply and that when the forest cover is replaced by small growth, the effect on flood crests is negligible. The investigations were made on small areas; nevertheless, the conclusions can be applied to larger streams and for areas differing widely in physical characteristics.

Extreme flood conditions in the streams flowing across the South Atlantic Coastal Plain are caused by tropical storms, and these storms are variable in route traversed, rapidity of movement, and quantity of rainfall. The forests, or lack of forests, are not factors of importance in the flood discharge. It is of interest to record the maximum discharge of the Savannah River at Augusta, Ga. This city is at the Falls of the Savannah and is on the western edge of the Coastal Plain. The water-shed up stream is a rolling plateau in which the streams occupy deep ravines. The plateau about the head-waters of the Savannah is broken by hills and low mountains. The virgin forests were cut long ago for lumber and the area is extensively farmed. There has been an approximately uniform increase in the area under cultivation, modified to some extent by the changed economic conditions following the Civil War between the States. The record for the Savannah River at Augusta, in extreme floods, is as follows:

Year	Maximum discharge, in cubic feet per second.	Year.	Maximum discharge, in cubic feet per second.
1796	360 000	1928	226 000
1852	255 000	Sept., 1929	374 000
1888	300 000	Oct., 1929	383 000
1908	307 000		

The record of this great river sustains the conclusion that the cutting away of the forests does not cause an increase in the flood discharge. In this connection, it is worth while to remember that a principal factor in the destruction of the small growth cover that follows the cutting of the virgin forests, is the policy of the forest conservationists of keeping annual fires from the cut-over forest. That policy permits the accumulation of débris on the forest floor from year to year. The accumulation soon provides fuel sufficient to produce a fire intense enough to destroy the smaller growth, and thus prevents the quick reforestation of lands unsuitable for cultivation. In the dry year, 1931, much of the young forest growth was destroyed in North Carolina, South Carolina, and Georgia, as a result of the application of the prevailing forest-conservation theories.

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[13] Chf. Engr., A. C. L. R. R., Wilmington, N. C.

A. L. SONDEREGGER,[14] M. AM. SOC. C. E. (by letter).—This paper forms a timely and valuable contribution to the knowledge of the function of the forest relative to stream flow. The writer agrees with the authors in their deductions, however, having clearly in mind that these deductions apply to water-sheds with residual, and, hence, with relatively shallow earth cover, where deep penetration of rainfall cannot take place. Possibly special stress should have been laid upon this feature, because water-sheds and forest growth on formations that originate in glacial moraine, or that consist of alluvial fills, may perform a function distinctly different from that described in the paper.

The writer is familiar with Fish Canyon water-shed and its steep slopes covered with residual soils, which probably average less than 8 ft. in thickness. It is also inferred from the description of Water-Sheds *A* and *B*, at Wagonwheel Gap, Colo., that the soil is not of sufficient depth to permit of deep penetration beyond the reach of roots. The statement[15] has been made that the augite-quartz-latite that comprises the entire foundation of the two watersheds breaks down, by reason of its fine, crystalline structure, into a rather fine and compact clayey loam. The report states, however, that because of the steepness of most of the water-shed slopes, this quality is only partly developed; that is, steady sheet erosion prevents the accumulation of deep masses of soil and causes rock fragments to comprise a considerable proportion of the mass, sometimes as much as 50% of the first 4 ft. of soil. Furthermore, erosion and leaching tend always to rid the soil of clay and silt, leaving the coarse sand. The result on all the steeper slopes is a quite permeable and well-drained soil layer, the depth of which has never been directly investigated on the main slopes. This description, the steep slopes, and the relatively young and slender tree growth, indicate a shallow earth cover.

The intensive development that has taken place in the semi-arid West during the 20 or more years since about 1910 has brought about, in many regions, a demand for water supplies in excess of those naturally available. It is realized that the evapo-transpiration requirements of the water-shed cover may consume from 40% to 100% of the seasonal precipitation, so that the run-off available at the foot of the range may be only a fraction of the meteoric supply. It has also become apparent that flood waste is, in many cases, only a relatively small portion of the total run-off, that the cost of its conservation is economically prohibitive, and, from the standpoint of water supply, that it is of minor importance.

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[14] Cons. Engr., Los Angeles, Calif.

[15] *Monthly Weather Review, Supplement 30*, p. 3.

Salvage of even a portion of evapo-transpiration losses being more promising, there have been, therefore, many proposals for the improvement of the water-shed with a view of increasing its water yield. Broadly speaking, they may be classed in two groups[16]:

First.—"Super-forestation" to the effect of producing a denser growth of native or other vegetation on the water-shed, in order (*a*) to "keep the water in the water-shed," as the saying goes; and (*b*) to induce more copious precipitation.

Second.—"Light deforestation" to restrict the vegetation to a minimum compatible with effective prevention of erosion, the intention being to diminish the evapo-transpiration requirements of the cover.

"Super-forestation" may be described as an effort to utilize the storage capacity of the water-shed to its maximum by increasing the absorption of precipitation and preventing flood waste by means of profuse vegetation. It presents, therefore, the logical solution for water-sheds overlying deep alluvial fills or glacial moraine, in regions enjoying abundant precipitation, where water supplies are required mainly for domestic purposes and depend upon underground supplies, springs, or unregulated stream flow. Forestation of bare water-sheds would have a similar beneficial effect under such conditions.

In the Middle Western and Middle Northern States, climatic conditions and the deep alluvial deposits of the plains or the moraines of one-time glaciated areas generally favor forestation as well as super-forestation. In semi-arid regions, super-forestation may be desirable if surface storage is lacking or flood run-off is abnormal because of a meager plant cover. Under the latter condition, however, the increased consumptive use of a densely grown water-shed cover has a tendency to reduce seepage run-off because, generally, vegetation is exceptionally tolerant. Vegetation is capable of sustaining growth in wet years almost as long as the moisture supply holds out, so that, in some measure, evapo-transpiration requirements may be proportionate to the density of vegetation.

It is quite apparent that super-forestation has its natural limitations because by natural selection the density of the existing vegetation, as a rule, is the one best adapted to the prevailing climatic conditions. Attempts to "make two trees grow where formerly grew one" over large areas, therefore, would have little chance of success. On the other hand, the forestation of bare areas overlying deep earth deposits unquestionably will promote percolation and increase ground-water supplies and normal stream flow at the expense, however, of a lesser total run-off. This result, under certain conditions, may also be desirable in semi-arid regions.

Practical proposals of super-forestation include the planting of trees in cut-over or burnt-over areas and the substitution of chaparral by trees. As a means of retarding surface run-off and inducing seepage, a dense growth of brush is likely to be as effective as a forest. Whether or not trees would grow in the chaparral zone may depend upon local conditions.

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[16] Unpublished report by the writer to the Water Conservation Committee of the Irrigation Division.

"Light deforestation" has been proposed with a view of reducing evapo-transpiration losses of the water-shed cover, and correspondingly increasing the water crop. It may present a logical solution for water-sheds having a shallow earth cover, in which roots penetrate to underlying bed-rock, and the seeped ground-water remains exposed to abstraction by plant growth until intercepted by surface drainage. Under such conditions, a large percentage, if not all, of the seepage is consumed by plant life, reducing the seepage run-off to only a fraction of the meteoric supply. In a general way, this was the condition prevailing in the water-sheds of Wagonwheel Gap, in Colorado, and of Fish Canyon, in Southern California.

The immediate effect of light deforestation is a greater surface run-off and lesser rainfall penetration, and the subsequent effect, a lesser evapo-transpiration loss. This result may be desirable in cases where the water demand exceeds the natural run-off and where storage of flood waste is practicable either in surface reservoirs or underground in the flood-plain. It applies to a large portion of the water-sheds of the semi-arid West.

Of the many proposals to effect light deforestation, the following deserve to be mentioned: The systematic burning of the underbrush of forested areas and of the chaparral of brush-covered water-sheds; the substitution of heavy water-using species of brush or trees by others of lesser consumptive use and the cutting of trees growing along watercourses, such as willows, alders, and sycamores. It is believed that measures of this kind would contribute to a conservation of water supplies. Whether the gain would justify the risk involved, or the destruction of the charm of the shaded brook, is quite another question.

Considering the matter strictly from the standpoint of conservation of water supply, both groups—super-forestation and light deforestation—have their merits, depending upon climatic conditions and the formation and depth of subsoils.

It may be of interest to note that super-forestation (also forestation) is constantly urged and promoted in the semi-arid Western regions with a view of increasing water supplies under conditions that are inimical to the success of the plan; that is, for steep mountain water-sheds with a shallow cover of residual soils and a mean rainfall less than the optimum use of the native vegetation.

Upon closer analysis it is found that the idea of super-forestation has been brought West, as an adjunct to the great process of interstate migration which has been so active during the twenty years (1912-1932), particularly from the prairie States. Most of the population of California, for example, settled in that State within the last generation, and having firmly in mind the benefits derived from forestation and super-forestation in their native Middle Western States, are now promoting the plan with remarkable tenacity, for conditions quite different. It is probable, for example, that the suggestion of replacing chaparral by forest originated with these people, under the claim that trees are better suited to keep the water in the water-shed and are inducive of more copious precipitation. The plan has been tried in Southern California, but failed because the trees would not grow without irrigation (except in a few favored spots).

From the water-supply standpoint, the plan has no merit: First, because the trees, like the brush, season after season, would consume the field capacity of the soil within the reach of their roots, with no chance of salvaging seepage water; and, second, because a fire (and fires do occur) would destroy the tree growth. Then, unless new trees were planted, the water-shed would remain bare until, in the course of years, a new growth of chaparral had returned it to its original condition. On the other hand, burned-over brush areas naturally recuperate within two or three years to the point at which the new growth, together with grasses and herbs, protects the soil from erosion. It is a fact that the ordinary brush fire neither destroys the roots nor burns up the seeds of herbs and grasses that seem to be buried in the soil, so that with the first rains an effective start is made toward the creation of a protective cover.

J. C. STEVENS,[17] M. AM. SOC. C. E. (by letter).—After many years of guessing and useless argument some authentic information has been produced at last as to the effect of forests on stream flow. The final conclusions are exceedingly simple. They might have been—in fact, actually were—anticipated many years ago. The forest, like every other vegetable crop, consumes large quantities of water for its growth. Unlike small plants, however, it also dissipates large quantities of water by mechanical means. It prevents a substantial portion of rain and snow from reaching the soil, permitting rapid evaporation from branches and leaves. When the forest is removed, the water thus consumed by it appears as run-off. As new growths appear the run-off gradually diminishes again in proportion to this crop and mechanical consumption. Two widely separate tracts cited by the authors under diverse climatic conditions attest emphatically to this fact. All other effects, such as advance or delay of maximums and minimums, are quite trivial, uncertain, and inconsequential.

Writers on this subject have been to exquisite pains to avoid the acknowledgment of the foregoing simple conclusion. Why all the feverish urge to prove that forests are intimately connected with an improvement of the regimen of streams? Why must simple facts be submerged under an avalanche of conjecture when they fail to meet preconceived specifications?

The answers are perhaps best stated by Dr. John Ise[18] in recounting the history of the Weeks Bill which was introduced in Congress in July, 1909, and passed in February, 1911. About $9 000 000 was appropriated for the purchase of forest lands in the Appalachian and White Mountains "for the protection of the water-sheds of navigable streams."

The debate over the Weeks Bill consumed one and one-half years of Congressional and National attention. It marked an epoch in the history of the forest policy in the United States. Dr. Ise declares that the main purpose of this Act in emphasizing the assumed relation of forest conservation on the navigability of streams, was merely to meet the question of constitutionality. He points out that there is no agreement among authorities as to the effect of forests in equalizing water flow; that the benefits have been greatly exaggerated by many writers; and that such benefits were grossly exaggerated in the debates in Congress.

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[17] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

[18] "The United States Forest Policy," by John Ise, Yale Univ. Press.

The following is an example of the gross exaggeration prevalent during the heyday of militant conservation:[19] "The connection between forests and rivers is like that between father and son. No forests, no rivers."

Another calamitous prophecy was to the effect that "when our forests are gone the streams will dry up, the rivers will cease to run, the rain will fall no more, and America will be a desert!"[20]

In 1910, the writer was employed on the famous "Smoke Case" by the Anaconda Copper Mining Company to make a study of the effect of forests on stream flow, climate, and soil, with particular reference to conditions in the Deer Lodge National Forest surrounding the huge Washoe Smelter, at Anaconda, Mont. This study was prompted by a suit instituted by the U. S. Forest Service to enjoin the Company from further operation of the smelter on the grounds that the smoke was destroying the surrounding forest and that the destruction of the forests would result in damages amounting to untold millions to the valley of Clark Fork River in increasing its floods, diminishing its minimums, and filling its valley with the soil from countless acres of denuded forest areas. The picture set up in the complaint was very gloomy.

The writer entered this study with an open mind, if anything, somewhat biased in favor of the forest-water-supply theory by reason of having absorbed most of the propaganda extant at the time. He gathered all the long-time stream-flow records in the world on which decided changes in forest cover have taken place. Among them were the Tennessee, Ohio, Ottawa, Murray (Australia), Merrimac, Sudbury, and Croton Rivers. No changes in regimen that by the greatest stretch of the imagination could be attributed to changes in the forest cover, could be detected. Any cyclic variations disclosed in run-off was accounted for readily and simply by similar variations in climatic factors. The generalized results of this study have been published.[21] After this study the writer felt fully convinced that the forest-water-supply theory belongs in the catalog of political expediencies rather than that of the sciences. Man's efforts to change the regimen of streams are puny and futile. The governing factors belong to forces far beyond his power to control.

Recently, the writer had occasion to go over a part of the lands around the Washoe Smelter, that were the subject of such intense study twenty-two years ago. The smoke has killed practically all the trees leeward of the smelter for many miles. Of course, Government and private owners have been amply compensated for their loss. The streams flowing from this graveyard of trees still run crystal clear and sparkling. There is no diminution in the supply as far as ocular evidence can be trusted. There is no more evidence of soil erosion than existed while the trees were alive and green.

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[19] "The Fight for Conservation," by Gifford Pinchot, Doubleday, Page & Co., New York, 1910.

[20] *Scrap Book*, September, 1908.

[21] "Forests and Their Effect on Climate, Water Supply, and Soil," by J. C. Stevens, M. Am. Soc. C. E., *Journal*, Assoc. of Eng. Societies, July, 1913.

Every one loves the forests; every one is glad they are in the hands of the Government and is grateful that they are being conserved for this and future generations. Their value for timber supplies, for recreational playgrounds, and to soothe æsthetic senses are ample and cogent reasons for preserving that control. If it was necessary once to attribute to them properties they do not possess, in order to "kid" the people and Congress into doing the thing that everybody wanted done anyway, that necessity has passed. Forest control is now a blessed fact. Is there any further necessity to keep up the farce?

HARRY F. BLANEY,[22] ASSOC. M. AM. SOC. C. E. (by letter).—Although European and American experimenters have been observing the influence of forests on stream flow for many years, no final conclusions covering the entire subject have been definitely established.

One needs only to review some of the voluminous literature on this subject to be convinced that there is a great difference of opinion among authorities. For instance, some believe that because rainfall is most abundant where forests grow, that they exert an important influence on the amount of precipitation, and that it is a definitely established fact that the destruction of forests is followed by a decrease in rainfall and drying up of streams and springs. Others have reached the conclusion[23] that "precipitation controls forestation but forestation has little or no effect upon precipitation. * * * The run-off of our rivers is not materially affected by any other factor than precipitation."

Thus, any new data that may throw additional light on this perplexing subject, are welcomed by the engineer, forester, and agriculturist. The authors are to be complimented on their excellent paper. The writer will confine his discussion to the part pertaining to Southern California, as most of his experience has been in that section.

Some investigators have been forced to the conclusion that "in respect to run-off, each stream is a law unto itself." This, undoubtedly, is true in many instances. There are so many complex factors influencing stream flow that the problem of determining the effect of forests on run-off by comparison of the discharge of streams of forested and denuded water-sheds is extremely difficult. Run-off is influenced so largely by the character of the precipitation, topography, elevation, geology, vegetation, and artificial control that quantitative measurements of the effect of forests on the flow of streams by comparing forested and deforested areas are usually open to criticism. However, the authors have adopted ingenious methods to overcome most of these objections.

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[22] Irrig. Engr., U. S. Dept. of Agriculture, Los Angeles, Calif.

[23] A report on "The Influence of Forests on Climate and on Floods," by Willis L. Moore, H. R., U. S. Committee of Agriculture, 1910, p. 37.

A discussion of "forests and stream flow" would not be complete without including the subject of consumptive use of water by native vegetation. That a gap exists between the quantity of rain falling on a given water-shed and the total run-off is clearly indicated by an analysis of data shown in Tables 2 and 3. For the period under consideration, the minimum annual run-off of the Santa Anita Creek occurred in 1924-25 and was 1.24 in., or about 6% of the total precipitation. In other words, the precipitation unaccounted for in 1924-25 was 20.67 in., or 94%; while in 1921-22, it was 30.62 in., or 51 per cent.

Whether this difference was due entirely to evaporation and transpiration losses is a debatable question. The authors state that it is doubtful whether any under-flow passes either station. Run-off, according to Meyer,[24] constitutes the residual precipitation after evaporation, transpiration, and deep seepage losses have been supplied, and the demands of evaporation and transpiration require from about 15 to 25 in. of precipitation per annum. Zon states[25] that the water consumed by the forest is nearly equal to the total annual precipitation—in cold and humid regions less, and in warm and dry regions somwhat greater.

In evaluating the evapo-transpiration losses of drainage basins, such as Santa Anita Creek and Fish Creek, it is desirable to divide the water-shed into two areas: (1) Drained slopes; and (2) canyon bottom. The "drained slopes" or sides of the canyon usually support chaparral, while in the "canyon bottom" water-loving trees, such as alders, willows, and sycamores, predominate. Evapo-transpiration losses from "drained slopes" start with the first rains in the fall and continue throughout the rainy season into the summer, until all the moisture in the soil, available for plant growth, has been extracted. It is difficult to determine these losses by field measurements. However, the consumptive use of water by brush cover on the valley floors has been successfully measured by soil-sampling methods.[26] Evapo-transpiration losses in the "canyon-bottom" area continue throughout the summer months, or as long as water is supplied from deep springs or is available from storage in alluvial fill. Under favorable conditions these losses can be measured.

The conservation of water by removing brush from the "drained slopes" is remote; the resulting damage by erosion would more than offset the value of the water saved. However, conservation by reducing the evaporation and transpiration losses in "canyon-bottom" areas of a water-shed is more promising. The City of San Bernardino, Calif., is accomplishing this by piping water out of the stream channel. In Southern California this expense is justifiable since title to water diverted by gravity from streams has a market value of from $2 000 to $4 000 per miner's in. (continuous flow), or $100 000 to $200 000 per sec-ft., depending on its use.

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[24] "Elements of Hydrology," by Adolph F. Meyer, M. Am. Soc. C. E.

[25] "Forests and Water in the Light of Scientific Investigation," by Raphael Zon, Forest Service, U. S. Dept. of Agriculture, 1927, p. 20.

[26] "Rainfall Penetration and Consumptive Use of Water in the Santa Ana River Valley and Coastal Plain," by Harry F. Blaney, Assoc. M. Soc. C. E., C. A. Taylor, and A. A. Young, Assoc. M. Am. Soc. C. E., *California State Bulletin No. 33.*

Little information is available as to the quantity of water lost in the bottom of canyons through evaporation and by transpiration from vegetation. In the spring of 1929 some experiments were conducted by the U. S. Bureau of Agricultural Engineering on the "consumptive use of water by native vegetation along stream channels"[26] in Temescal Canyon, 4 miles southeast of Corona, Calif. The vegetation consisted of willows, tules, and kindred moist land growths. For a 30-day period, from April 28 to May 27, inclusive, the total evapo-transpiration was 12.9 acre-in. per acre.

In 1931, this investigation was extended to Coldwater Canyon[27] near Arrowhead Springs, Calif., and methods were developed to measure the evaporation and transpiration losses in canyon bottoms.

One of the methods used was to measure losses of stream flow between two bed-rock controls, located about 2 100 ft. apart in the canyon. The approximate elevations of these controls above sea level are 2 300 and 2 500 ft., respectively. The canyon between the controls is narrow and has precipitous sides, with the bottom width ranging from 30 to 50 ft., and the vegetation is mostly alders, with some sycamores, bay, oak, and herbaceous growth. During the period of record, there were no visible indications of water reaching the section under study other than that passing the upper control. At each control a low concrete dam was built across the stream channel on a bed-rock foundation. A 3-in. Parshall measuring flume, equipped with a Stevens type L water-level recorder, was placed at one end of each dam for the purpose of measuring the flow of water passing into and out of the experimental section of the canyon between the controls. At first, values for the gauge height for each hour were used in computing the volume of water passing the two controls. The daily loss between controls was obtained by subtracting the daily volume of water passing the lower control from the daily volume passing the upper control. This proved to be a lengthy and laborious process, and in order to eliminate a large part of this routine work, a flow recorder was attached to the water-stage recorder. This flow-recorder attachment consists essentially of a spiral cam that solves the flow formula mechanically.

The loss of water per day between the upper and lower controls may be obtained directly by superimposing the two flow-recorder charts, one from each control, and measuring the area between the two curves by planimeter. This area represents the daily loss between the controls, and when multiplied by the proper constant can be converted into the hydraulic units desired.

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[27] An unpublished Progress Report on Evaporation and Transpiration Studies in Coldwater Canyon, by Harry F. Blaney, Assoc. M. Am. Soc. C. E., C. A. Taylor, and H. G. Nickle, Jun. Am. Soc. C. E., Bureau of Agricultural Eng., U. S. Dept. of Agriculture, 1932.

The relative amounts of the indicated losses that are chargeable separately to evaporation and transpiration are difficult to determine. However, Rohwer[28] has indicated that the rate of evaporation is relatively low when the temperature of the water is below that of the overlying air. This condition exists in Coldwater Canyon throughout the growing season as the water comes from cool springs and is shaded as it flows through the stream channel of the canyon. Thus, evaporation is small, and the loss indicated by the daily drop in stream flow is due principally to transpiration.

From the Coldwater Canyon measurements it is estimated[27] that the consumptive use of water by canyon-bottom vegetation during the 6-months summer season, May to October, was 45 acre-in. per acre, 238 acre-in. per mile of canyon, or a depth of 0.10 in. over the water-shed per mile of canyon.

Referring to data given by Messrs. Hoyt and Troxell, the average run-off from Santa Anita Canyon during the six summer months, May to October, inclusive, for seven years ending September, 1924, was 1.19 in. in depth over the entire water-shed. The stream in the Santa Anita Canyon (including the tributaries) is estimated to be 13 miles long. Assuming that the data obtained from the Coldwater Canyon experiment can be applied to Santa Anita Canyon, the use of water by canyon-bottom vegetation would be 1.30 in. in the latter. This amounts to 109% of the average run-off for the same period or, in other words, the run-off for the six summer months would be increased 109% if no water was used by the canyon-bottom vegetation.

In like manner, an estimate can be made of the summer consumptive use of water in Fish Canyon. From the Fish Creek water-shed for the 6-months' period—May to October—for seven years ending September, 1924, the average run-off in depth, in inches, was 0.90. The stream is estimated to be 10 miles in length, and the use of water by canyon-bottom vegetation, 1.0 in., or 111% of the average run-off for the 6-months' summer period for seven years preceding the fire. This result is somewhat surprising since the authors show an annual run-off of only 1.00 in. in Fish Canyon the year before the fire, and an annual normal run-off of 1.07 in. the year following the fire. In other words, for these years the summer use of water by canyon-bottom vegetation was 100% of the total annual run-off. The increased run-off for the six months during the summer following the fire was only 0.40 in. Thus, if all the water used by canyon-bottom vegetation during the summer months could be conserved by piping it out of the stream channel, the increased run-off would be 1.0 in., or 250% of the increased run-off resulting from the burning off of the water-shed. However, it is doubtful whether more than 80% of the water consumed by the canyon-bottom vegetation could be saved.

In closing, the writer would like to call attention to the serious problem of erosion in Southern California. The data given in Table 12 of the paper illustrates clearly the large quantity of silt carried by a stream after the vegetation has been burned off a small water-shed. It should be pointed out that the rainfall that year was about 30% below normal; and, thus, only a minimum of damage was done. However, in a year like 1921-22, with an annual precipitation of 60 in., or twice the normal, the amount of damage due to erosion would have been tremendous if the chaparral cover had not been there to prevent it.

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[28] "Evaporation from Free Water Surfaces," by Carl Rohwer Assoc. M. Am. Soc. C. E., *Technical Bulletin No. 271,* U. S. Dept. of Agriculture, December, 1931.

DANIEL W. MEAD,[29] HON. M. AM. SOC. C. E. (by letter).—This excellent paper presents the most exact and conclusive measurements that are available of the effect of deforestation and denudation of drainage areas on the flow of streams. The only uncertainties in the conclusions as regards the streams discussed seem to be in the relative quantity and distribution of the rainfall on the areas of the Santa Anita and Fish Creeks in Southern California, and the possible different effects of such variation of rainfall on the comparative flow of these streams before and after the time when deforestation and denudation occurred. It will be noted that the only rainfall stations available for these areas are at Mount Wilson and at the Santa Anita Ranger Station, both of which are west of Fish Creek and cannot represent the rainfall accurately on that drainage area which, in each rainstorm, would probably vary somewhat from the rainfall at these stations just as the rainfalls at the stations vary one from the other. The increase in flow from the drainage area of Fish Creek, however, is so considerable that there can be no question as to the actual increase in flow resulting from the deforestation and denudation of the Fish Creek drainage area, although the actual quantity seems somewhat uncertain.

The rainfall measurements on the Wagonwheel Gap areas are much more complete and represent the actual conditions as well as they could reasonably be determined. In the writer's opinion there can be no doubt as to the essential validity of the general conclusions that will be drawn from these data, that, in general, increased flow will follow deforestation and denudation on small drainage areas where other factors remain the same.

It is obvious that: (1) The relative results of deforestation or reforestation in each case must depend upon the physical and climatic factors that prevail on the areas from which such results obtain; (2) in applying these data to other small areas, the comparative physical and climatic factors of such areas must be considered; and (3) any variation in such factors will necessarily modify the results to be expected.

In extreme and rare cases it is possible that deforestation, and especially denudation, of small drainage areas might even reverse the results obtained under the conditions considered by the authors.

It is particularly worthy of note that the drainage areas considered in the paper are comparatively small (Area *B* of Wagonwheel Gap, 200.4 acres; and area of Fish Creek, 6.50 sq. miles), and that while the results of deforestation and denudation are quite marked on these limited areas, it is obvious that, as affecting the larger drainage areas of which they are a part, the results might be quite insignificant and indistinguishable. Therefore, it is still necessary to utilize inductive reasoning in discussing the application of the data from these small areas to the larger areas of great rivers.

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[29] Prof. Emeritus, Hydr. and San. Eng., Univ. of Wisconsin; Cons. Engr., Madison, Wis.

As a basis for such reasoning, it is necessary to consider first principles as they are demonstrated by the simplest experiments and examples that are available.

Rainfall.—It is quite obvious that the principal factor that affects stream flow is the quantity and distribution of the rainfall on its drainage area. Evaporation (including plant transpiration), percolation, and run-off are dependent on the physical and climatic conditions on each drainage area. Each increases in general as rainfall increases, although not in direct proportion. The departure from direct proportion; that is, the difference between rainfall and evaporation, percolation, or run-off measures the effect of temperature, humidity, wind velocity, distribution of rainfall, and other minor factors. When the rainfall occurs in light showers, the proportion lost in evaporation and transpiration is greatly increased; and, in most places in the United States, if the annual rainfall were distributed equally over each day of the year, it is doubtful whether any of it would flow away in streams unless it accumulated as snow or ice during the winter and melted suddenly on the advance of spring. With heavy showers and torrential rainfall, run-off and percolation (on pervious soil) increase and stream flow is rapidly augmented.

Effects of Geology and Topography.—There can be no question but that, next to intensity of rainfall, the geology and topography of a drainage area are the principal factors that control the proportion of rainfall that appears as stream flow. Fundamentally, the character of bed-rock of a drainage area is important. In Wisconsin, the rainfall conditions throughout the State are fairly constant, and the average is about 31 in. An investigation made a few years ago[30] showed that the average annual run-off from crystalline rock areas was 13.09 in.; from sandstone areas, 10.27 in.; and from limestone areas, 9.39 in. In some places, these areas are deeply covered with glacial drift (200 ft., or more), so that the bed-rock effect is modified considerably by pervious and impervious mantle deposits as well as by surface slopes and surface conditions, such as forests, cut-over land, cultivated land, swamps, lakes, etc. In fact, the conditions of the mantle deposits, soils, and subsoils lying over the bed-rocks may have as great an effect on run-off—and especially on the unformity of flow—as the bed-rock.

For example, the average yearly run-off of the Manistee River, at Sherman, Mich. (1904 to 1915, inclusive), was about 17.1 in., with an average rainfall of about 29.5 in., which is 1 or 2 in. less than that for the State of Wisconsin. The bed-rock is about one-half sandstone and one-half shales, and the high average annual flow, greater than that from the crystalline area in Wisconsin, is due to the deep sandy soil and subsoil over the bed-rock that rapidly absorbs rainfall and reduces evaporation and transpiration. On account of the storage in the great bed of sand, the flow, in cubic feet per second per square mile, is far more uniform, with an average of 1.26, a maximum of 3.89, and a minimum of 0.67.

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[30] "The Effects of Bedrock on Runoff of Wisconsin Streams," by E. E. Foster, *Proceedings*, Eng. Soc. of Wisconsin, 1926, p. 162.

Any fragmentary mantle material that will remove the rain water promptly from the surface and store it at depths beyond the reach of evaporation will produce a similar effect on run-off. For example, the Deschutes River, of Oregon, has volcanic lava rock as its bed-rock. The upper part of this rock is cracked and fissured at the surface and perhaps to a depth of 1 000 ft., or more (its canyon in places is 2 000 ft. deep). It is covered by a comparatively thin and pervious soil over much of its area. From the limited data available the writer would judge that the average annual rainfall for the area probably does not exceed 12 in. The annual flow from this area averages about 8.75 in., and the flow, in cubic feet per second per square mile, has an average of 0.645, a 19-year maximum of 4.85, and a minimum of 0.38. It appears from the foregoing that the nature of the bed-rock and its pervious condition may have much to do with stream flow, but that pervious mantle deposits and fissured rock in the structure above the stream bed giving ample storage are the greatest factors in low floods and uniform flow of streams.

Surface and Soil Conditions.—Where pervious mantle deposits exist on a drainage area the disposition of the rainfall is greatly modified by the nature of the deposit and by surface conditions. A knowledge of the rainfall disposition has been of much importance in agriculture, and many experiments have been made to determine the evaporation and precolation through various classes and depths of soils and under various surface conditions. The writer has collected and analyzed the results of these experiments in another place.[31]

The points particularly noted[32] were the high evaporation and small percolation from sod, the high and maximum seepage through cultivated ground, and the high percolation and small evaporation from sand and pervious soils.

The run-off from any area is dependent on the pervious or impervious condition of the soil, the surface condition or covering, and the slope. Experiments were made in Wisconsin in 1929[33] on the run-off from various land areas. The highest average percentage of run-off found was less than 2% for forest covering, 20.1% for pastures, 24.1% for oat lands, 25.6% for corn land, and 21.5% for hay lands, the last three being probably somewhat reduced by the less average slope. With soil classed as sandy, the run-off in most cases was reduced below the average, undoubtedly by increased percolation. Here, again, the actual soil conditions are somewhat indefinite and the nature of the soil involved is not clearly discernible.

One of the most noteworthy examples of the increase in evaporation and transpiration (and possibly of percolation), and of the consequent reduction in run-off, is found in the change in the condition of Devils Lake, in North

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[31] "Hydrology," by Daniel W. Mead, Hon. M. Am. Soc. C. E., McGraw-Hill Book Co., p. 138 *et seq.*

[32] *Loc. cit.*, Fig. 84, p. 139.

[33] *Bulletin 99*, Agricultural Experiment Station, Univ. of Wisconsin, Wis.

Dakota (see Fig. 10). This lake, which has no apparent outlet and is very alkaline, had an area of about 115 sq. miles as determined by the U. S. Land Survey in 1883, and was at Elevation 1 435 (corrected elevation). In 1883, the surrounding country was opened to settlement. While the average annual rainfall in this area has been about 18 in. (see Fig. 11), the run-off (about 1 in. per annum) from the original prairie land surrounding the lake was sufficient to maintain it at about Elevation 1435 although it evidently fluctuated considerably with cycles of high and low rainfall years. (Note in Fig. 11 the reduction in elevation from 1867 to 1879). Since the land around the lake has been settled and cultivated, there has been a considerable decrease in run-off and a marked shrinkage in the height and area of the lake, evidently due to the opening of the land by cultivation. In 1927, its surface had fallen so far that it became divided into six or more smaller lakes and ponds, with an area of about 45 sq. miles and an elevation 21 ft. below that of 1883. At present (1932), the lake has fallen about 3 ft. more, or to Elevation 1 411, and has an area of approximately 40 sq. miles.[34]

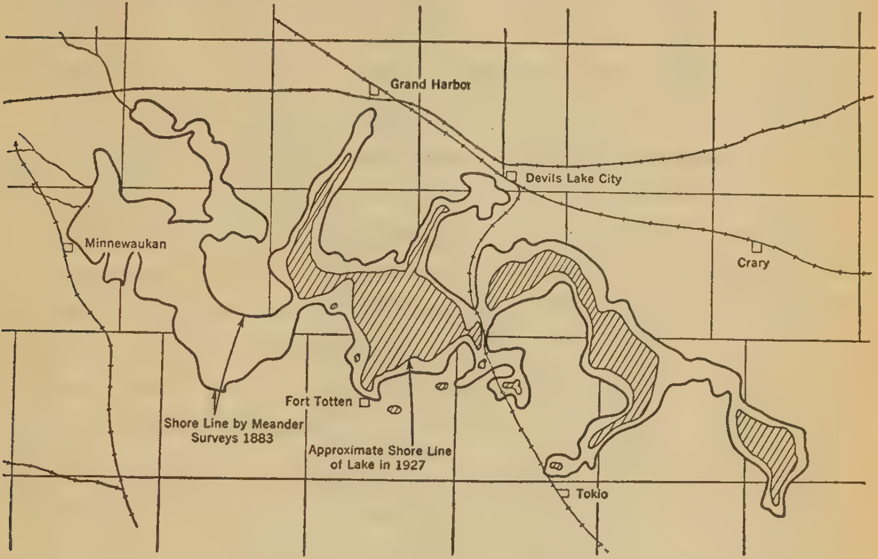
Grand Harbor
Devils Lake City
Minnewaukan
Crary
Fort Totten
Shore Line by Meander Surveys 1883
Approximate Shore Line of Lake in 1927
Tokio

FIG. 10.—DEVIL'S LAKE, NORTH DAKOTA.

A similar explanation will account for the drying up of certain springs, which has been attributed to deforestation. The probability of this assumption seems to be borne out by the data given in Table 13.[35] It will be noted from Table 13 that the daily consumption of water by most crops is given as several times greater than that of oak and fir trees. On the other hand, Harrington[35] estimates that on an average only 70% of the rainfall reaches the ground under a forest covering, and that 30% is re-evaporated from the trees and their foliage.

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[34] Information furnished by E. F. Chandler, M. Am. Soc. C. E.

[35] Rept., Kansas State Board of Agriculture, December, 1889.

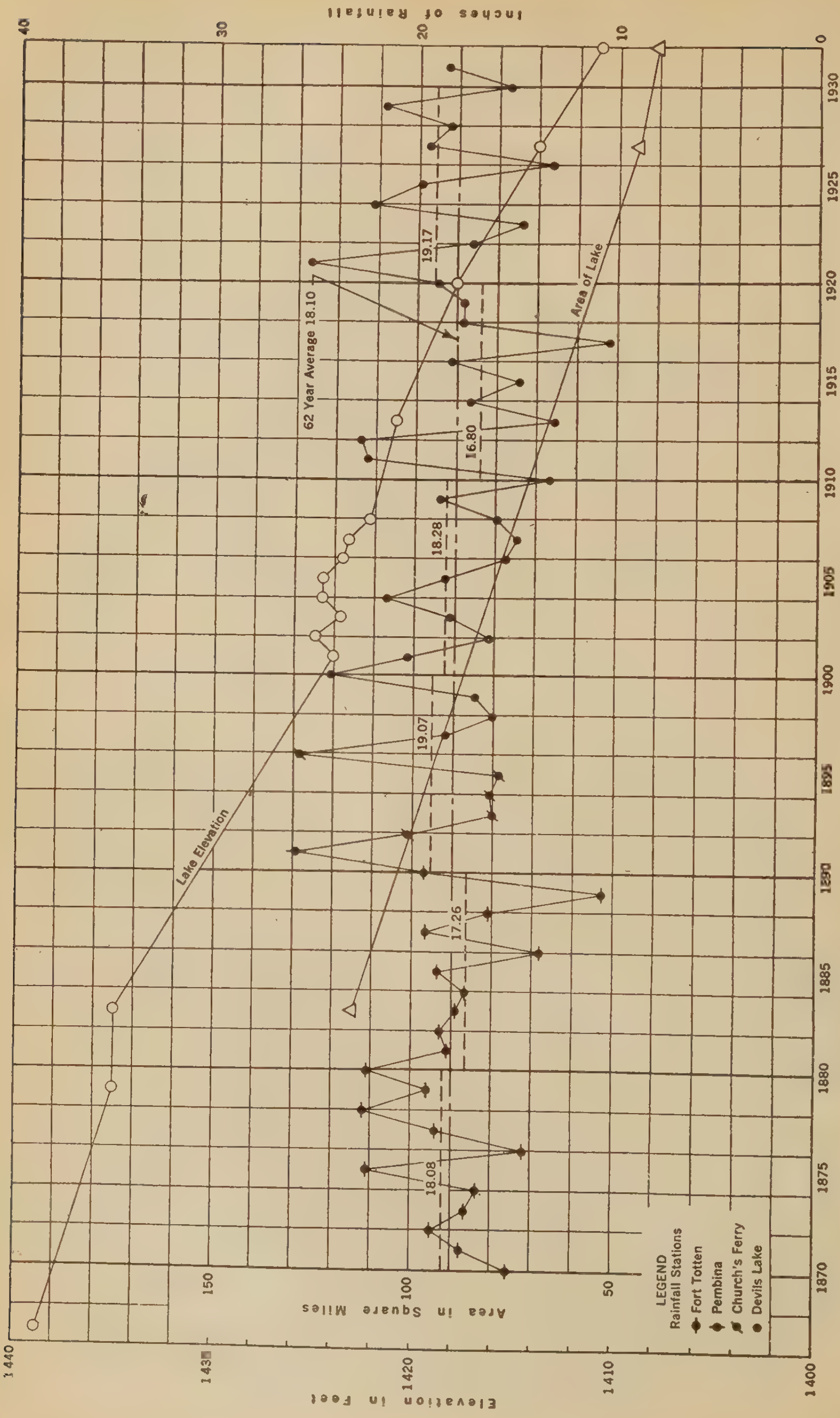

FIG. 11.—RAINFALL DATA AND LAKE LEVELS, DEVIL'S LAKE, NORTH DAKOTA.

TABLE 13.—DAILY CONSUMPTION OF WATER BY CROPS

Crops	Inches of Water		Crops	Inches of Water	
	Minimum	Maximum		Minimum	Maximum
Lucern grass	0.134	0.267	Wheat	0.106	0.110
Meadow grass	0.122	0.287	Rye	0.091	.....
Oats	0.140	0.193	Potatoes	0.038	0.055
Indian corn	0.110	1.570	Oak trees	0.030	0.038
Clover	0.140	.....	Fir trees	0.020	0.043
Vineyard	0.035	0.031			

The total use of water by trees compared with the evaporation from open water is given, as follows:

Transpiration	77%
Re-evaporation	61%
Forest soil evaporation	13%
Total percentage of open-water evaporation	151%

Woolny[35] has estimated the return of water to the atmosphere by various kinds of vegetation, compared with evaporation from open water, as given in Table 14.

TABLE 14.—PROPORTION OF WATER RETURNED TO ATMOSPHERE AS COMPARED WITH EVAPORATION FROM OPEN WATER

Vegetation covering	Proportion of water returned, percentage	Vegetation covering	Proportion of water returned, percentage
Sod	192	Mixed crops	144
Small grass	173	Bare soil	60
Forest	151		

Raphael Zon, Director of the Lake States Forest Experimental Station, considers[36] forests as the greatest evaporators of water, exceeding all other vegetable coverings. He quotes Otozky, a Russian physicist, as estimating the amount of transpiration from forests as nearly equal to annual precipitation, and is of the opinion that if the Atlantic plain and the Appalachian regions were deforested it would have a considerable influence on the humidity and rainfall of the Central States and prairie regions to the west. If this is true, it is difficult to see forests as conservers of waters for springs and stream flow. Tables 13 and 14 indicate entirely different conclusions and show that deforestation, followed with replacement by meadows and cultivated ground, will account for decreased run-off if such decrease were observed.

With the foregoing facts in mind, the writer believes that it is both logical and practical to outline the conditions favorable or necessary to assure certain conditions of run-off. This has been attempted for two extreme and opposite conditions, as follows:

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[36] *Science*, Vol. 38, p. 71.

I.—Conditions Favorable to Maximum Annual Discharge and Equalized Stream Flow with a Given Rainfall:

(*a*) A fairly deep impervious basin underlying the drainage area from which no water can escape, except through the normal outlet of the stream.

(*b*) The basin in Condition (*a*) filled with pervious material, such as sand, sandstone, fractured volcanic or other cracked and fissured rocks (such rocks being covered at the surface with pervious soils), of such a nature and depth that the rains falling on the surface will sink rapidly into storage in the basin. The water must settle fast enough to avoid excessive surface evaporation and deep enough so that capillary attraction will not draw the stored water to the surface.

(*c*) The basin in Condition (*a*) with no surface storage on the drainage area.

(*d*) The basin in Condition (*a*) with only slight surface slope so that there will be no surface run-off, but rapid absorption into the pervious fill.

(*e*) A basin in which the area is entirely devoid of vegetation.

II.—Conditions Favorable to Torrential Flow and Maximum Variation in the Continuity of Flow with a Given Rainfall:

(*a*) An impervious basin, with steep channel and abrupt slopes to the divide; and,

(*b*) An impervious basin, with unobstructed channel and no mantle deposits or vegetation.

Under Conditions I, the rain falling on such a drainage area will sink rapidly into the soil and pervious underlying deposits, thus avoiding surface evaporation, and flow slowly toward the river channel, being sufficiently detained in storage to equalize the flow partly between the more or less periodic rain storms that occur over the area. Under Conditions II the rains will flow rapidly to the channel and through the river outlet in maximum floods followed by periods of entire cessation of channel flow.

In both the foregoing cases the distribution, intensity, and quantity of rainfall will have marked effect on the flow of the stream, but under the conditions of rainfall prevailing, one case will result in the best possible regulation with minimum floods and the other in maximum irregularity and maximum floods.

With these ideal conditions representing the extreme of regularity and irregularity in stream flow, almost any change in the conditions on these two areas will necessarily have opposite effects. In the first case, if the valley is covered with forests and vegetation, the pervious surface becomes more or less filled with roots and, therefore, less pervious. Evaporation will be increased, plant life will take from the supply, and flow will become more irregular and less in amount. In the second case, with the soil and detritus necessary for forest and vegetable growth, flow is obstructed, some storage is supplied, flood peaks will be reduced, and total discharge will be decreased.

Quantitative measurements of the actual effects of minor factors are frequently impossible although some effect may easily be demonstrable. Footprints across an impervious clay drainage area will fill with rain, reduce run-off by the amount so held, and will add to evaporation and possibly to seepage. Such results are positive although indistinguishable and unmeasurable as affecting stream flow.

Perhaps a more striking example of what would seem to be an important factor as affecting stream flow is shown in the apparent lack of effect on the drainage of large areas of land on the flood flow of the stream from the drainage area. Professors Woodward and Nagler[37] found, in the case of the Des Moines and Iowa Rivers, where a large portion of the drainage areas had been artifically drained, that during flood periods there were no significant changes in the behavior of the rivers that might be attributed to drainage.

The hydraulic engineer is perhaps more deeply interested in the question under discussion than are the men of almost any other profession. The conservation of water for water supply, water power, navigation, irrigation, etc., and the question of maximum floods in drainage and flood protection works are of the greatest importance. If water can be conserved, supplies increased, and floods prevented by any reasonable amount of forest covering, it is most important for him to know and to appreciate such facts.

Numerous investigations have been made by engineers interested in this question and unbiased in their opinion, and some of the results of such investigations are worthy of brief mention in this connection. Perhaps the most noteworthy of these studies was that of the late H. M. Chittenden, M. Am. Soc. C. E., already referred to by the authors.

In 1910, the writer endeavored to determine what influence, if any, the cutting of Wisconsin forests had had on the flow of Wisconsin streams.[38] No long-time measurements of stream flow were available, but from the gauge-height measurements of the U. S. Weather Bureau and the U. S. Army Engineers, the annual and progressive means of the annuals were found for the Wisconsin and Upper Mississippi Rivers.

The diagrams[39] clearly showed that high, mean, and low waters followed the quantity and distribution of the rainfall, and that there had been no marked change in the normal height of floods or in extreme low-water elevation.

In an investigation of the relation of forests to stream flow on the Tennessee and Cumberland Rivers, W. W. Harts, Brig.-Gen., U. S. A., M. Am. Soc. C. E., found[40] the relations of the height of high and low water at various periods during and following deforestation to be as shown in Table 15. He estimated that, in 1909, 60% of the drainage area of the Tennessee River was forested and that 20% of the area had been cleared in the preceding twenty-five years.

While these data (Table 15) might be taken to indicate a greater prevalence of extreme high and extreme low water during the earlier period when a greater proportion of forest covering existed on the drainage area, General Harts stated that the frequency and height of high water in both these rivers, as well as the duration of low water, followed the rule of precipitation closely enough during the period under observation to force the conclusions that this is the principal cause of the variation in these streams, and that "the introduction of forestry as an important factor in either direction is forced and illogical."

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[37] "The Effect of Agricultural Drainage upon Flood Run-Off," by Sherman M. Woodward and Floyd A. Nagler, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 821.

[38] "The Flow of Streams and the Factors That Modify It," by Daniel W. Mead, Hon. M. Am. Soc. C. E., *Bulletin No. 425,* Univ. of Wisconsin.

[39] *Loc. cit.,* Figs. 68 to 70, p. 308.

[40] *Professional Memoirs,* U. S. Corps of Engrs., Vol. I, 1909, S. P. 937, p. 397.

TABLE 15.—Relations of the Height of High and Low Water During and Following Deforestation

Period	Tennessee River		Cumberland River		
	Gauge height, in feet	Average number of days per year	Gauge height; in feet	Total number of days	Average number of days per year
High-Water Stages					
1899–1908...........	>25	7.0	>35	78	.....
1889–1898...........	>25	9.1	>35	123	.....
1879–1888...........	>25	11.6	>35	178	.....
Low-Water Stages					
1899–1904...........	<1	10.3	<1	.....	20.8
1889–1898...........	<1	7.7	<1	.....	32.0
1879–1888...........	<1	20.4	<1	.....	32.5
1899–1904...........	.....	.....	<5	.....	161.0
1889–1898...........	.....	.....	<5	.....	153.8
1879–1888...........	.....	.....	<5	.....	141.8

The investigations by General Harts were based on areas, in which deforestation had taken place through the cutting of timber through a considerable term of years.

Edward Burr, Col., Corps of Engineers U. S. A., (*Retired*), M. Am. Soc. C. E., investigated the result of reforestation in the Merrimac River Basin on the flow of that river.[41] In regard to the forest conditions, Colonel Burr states that (1) deforestation continued progressively from the earliest settlement to about 1860-70; (2) reforestation through natural processes progressed from 1870 to 1910 at a rate that has increased from 1880 to 1900, or later; and (3) forest areas were larger in 1910 than in 1860-1870 by as much as 25 to 30% of the entire basin. Perhaps better data (longer and more accurate) existed for this area as regards rainfall and stream flow than on any other for which similar studies have been made.

These studies, which seem to have been very thorough, included investigation of the relation of precipitation to high, mean, and low-water conditions. As in other studies, the irregularity of the consecutive annual occurrence of rainfall and run-off is so great that progressive averages are needed in order to ascertain tendencies in their relations, and, in these cases, 10-year progressive averages were used. Colonel Burr calls attention to the fact that his diagrams for total annual days of flood and for days of flood per inch of precipitation are approximately parallel to the precipitation diagram, indicating quite clearly that, when eccentricities are eliminated by averages over a period of 10 years or more, the duration of floods is dependent largely upon precipitation and that the duration of floods per inch of precipitation increases with the latter. Furthermore, he stated that the same characteristics are recognizable in regard to annual discharge, annual flood discharge, the percentage of annual discharge that runs off during flood stages, and the flood discharge per inch of precipitation. These characteristics are so marked that it is difficult if not impracticable to eliminate the influence of precipitation and to ascertain the existence and effect of any minor influence, such as variable forest conditions.

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[41] See House Doc. No. 7, 62d Cong., First Session, May 23, 1910; also, *Engineering News*, Vol. 66, p. 100 *et seq.*

Colonel Burr found no evidence that deforestation or reforestation had modified the stream flow of the Merrimac River in any way. In this connection it seems pertinent to call attention to the fact that the highest floods that have ever been recorded on the Mississippi River, at St. Louis, Mo., were in 1785 and 1844, long before deforestation could have had any effect on the flow of that river.

These investigations and the data considered point to the conclusion that: (*a*) On large drainage areas any acts of man except the construction of relatively large storage reservoirs or extensive flood protection works, are so limited in extent that they can have little influence on the flow of streams; (*b*) stream flow follows rainfall; and (*c*), large floods are due to torrential rains, together with other physical and climatic conditions favorable to flood conditions. On small areas the cultivation of the land and the existence of extensive forests may have a considerable effect in decreasing the flow of small streams draining such areas, except that such effects are so small that they are likely to become insignificant and indeterminate on the flow of the main streams to which these small streams are tributary.

In general, there can be no doubt that all the factors described in this discussion have some effect on the disposition of rainfall, and, therefore, on run-off, but unless the effect is determinable and measurable it is theoretical and of little practical importance.

This subject has been before the public for many years and has been so surrounded by tradition and superstition that it has become greatly obscured. Extravagant claims as to the value of forests as a means of flood protection still persist. The lengthy discussion of this subject grows wearisome and would seem to be quite unnecessary were it not for its possible influence on legislation and the possibility of resulting large and useless (as far as flood protection is concerned) public expenditures. As late as February, 1929, the then Secretary of Agriculture, in his letter of transmittal to the President of the report on "Relations of Forestry to the Control of Floods in the Mississippi Valley," states:[42]

"Added study * * * shows that the forests of the Mississippi watershed were responsible for a reduction in the possible flood crest of nearly 15 inches.

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[42] House Doc. No. 573, 70th Cong., Second Session, pp. III and IV.

Furthermore were all the forests of the Mississippi valley properly protected and managed in accordance with established forestry principles and practice a further reduction in possible flood crest of 55 inches would be possible."

He further states that this retaining effect would be equal to a reservoir storage of 46 000 000 acre-ft.

Such statements are so palpably absurd that they might be ignored were it not for the fact that they form the basis for a proposed expenditure of about $2 000 000 per year for protection and extension (?) of the forests in the Mississippi Basin.

RALPH R. RANDELL,[43] M. AM. SOC. C. E. (by letter).—On the only two water-sheds for which the matter has been investigated conclusively, it was found that: (1) Forests and brush substantially lessen stream flow at practically all seasons and stages, including flood, minimum, total, and average flows; and (2) forests and brush substantially lessen erosion.

These relationships have been established experimentally as to Area *B* of the Wagonwheel Gap experiment, in Colorado, and the Fish Creek water-shed, in Southern California. They were demonstrated during each of seven consecutive years in the case of the Wagonwheel Gap water-shed, and six in the case of Fish Creek, a total of thirteen years. No year showed contrary results with either water-shed.

The Colorado and Southern California water-sheds are widely different geographically, climatically, topographically, geologically, and in soil and vegetative cover, form of precipitation, and regimen of run-off. Especially significant is the fact that the winter precipitation on the Wagonwheel Gap water-shed falls in the form of snow, remains frozen on the ground all winter, and does not run off as stream flow until melted by the rising temperatures of the ensuing spring. The precipitation over the Fish Creek water-shed, on the other hand, falls as rain, and runs off immediately, except in so far as it evaporates or seeps underground temporarily.

The fact that all conclusive investigations show the foregoing relationships, even under such widely different circumstances and over such long periods of time, presumptively establishes that these relationships are of general application, and are not merely the result of any local or temporary peculiarities.

The same laws necessarily apply also to large water-sheds, as well as small ones; because large water-sheds are only combinations of small ones, and, therefore, must inevitably re-act to given influences, such as vegetative cover, in the same manner as the small ones of which they are composed.

The authors' contribution consists of the presentation and original analysis of the Fish Creek data, and a re-analysis of the Wagonwheel Gap data along lines somewhat different from those used by Messrs. Bates and Henry,[7] but with the same results. Their excellent paper should be of great benefit in directing attention to the real facts as to the influence of forests on stream flow. It is high time for engineers to discard, once and for all, the hoary fallacies that have been propagandized so widely and persistently during the last score or more of years. Especially important is it that they realize at last that all conclusive investigations show that forests and brush do not increase, but instead considerably lessen, both total water supply and low-season stream flow.

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[43] Senior Engr., Federal Power Comm., Washington, D. C.

H. K. Barrows,[44] M. Am. Soc. C. E. (by letter).—A paper such as this is always welcome, adding as it does to the present information on a somewhat controversial subject.

The writer has studied the data presented from the viewpoint of water losses. It is obvious from Fig. 12 that on both the Wagonwheel Gap and Fish Creek drainage areas the yearly water losses have been decreased on the deforested or burned areas, and, hence, the run-off of these areas—which is the precipitation minus water losses—has been increased.

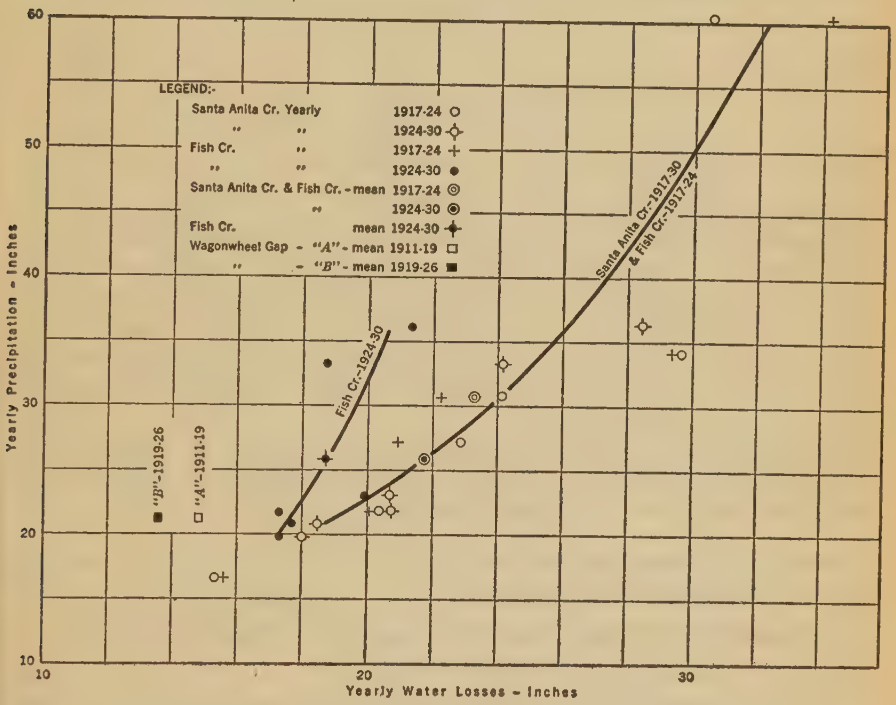

Fig. 12.—Comparison of Precipitation and Water Losses.

The lessening of water losses in respect to precipitation for the Southern California area is approximately as shown in Table 16.

The Wagonwheel Gap observations cover a much narrower range of precipitation (from about 17 to 24 in. yearly), but the period means for about 21 in. of rainfall, shown in Table 16, indicate fairly consistent results.

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[44] Prof. of Hydr. Eng., Mass. Inst. Tech.; Cons. Engr., Boston, Mass.

It is evident that, in many cases, the lesser soil evaporation upon forested areas is more than offset by the demands of the forest for transpiration as well as its interception losses. The net result is that the forested area may show greater water losses and, hence, less yearly run-off than the deforested area, under conditions otherwise similar.

TABLE 16.—WATER LOSSES

Yearly precipitation, in inches (1)	WATER LOSSES, IN INCHES		Column (2) minus Column (3), in inches (4)	Ratio: Column (4) / Column (2) (5)
	Before deforestation (2)	After deforestation (3)		
SOUTHERN CALIFORNIA AREAS				
20	18	17+	1	0.05
30	24	20	4	0.17
40	27	21	6	0.22
COLORADO AREAS				
21	15	13.5	1.5	0.10

There is much misunderstanding on this matter by the layman, and many water-works officials in the Northeastern United States are devoting time and money to tree planting on drainage areas used for water supply, with the idea that they are thus increasing the supply, when they are more probably diminishing it. It may be worth while for other reasons, and perhaps a good investment, to adopt such programs of tree planting, but certainly as regards increasing the water supply the results are likely to be detrimental rather than helpful.

More and better information is needed on this subject in other sections of the country and, if possible, some experimental work in the study of comparative water losses on forested and open areas.

DONALD M. BAKER,[45] M. AM. SOC. C. E. (by letter).—The basic equation covering the disposition of precipitation is, as follows:

$$\text{Precipitation} = \text{Evaporation} + \text{Transpiration} + \text{Surficial Run-Off} + \text{Ground Storage}$$

In a closed water-shed, ground storage usually appears as stream flow. Any physical change on a water-shed affecting either the right or left-hand part of this equation will likewise affect the other part. Precipitation that reaches the ground surface might be increased by deforestation through the elimination of interception by tree or brush growth. The extent of this interception, however, is small, except in light showers or snowfall, and is probably balanced or equalized by greater evaporation from the ground surface.

Assuming little or no increase in precipitation, increase in surficial run-off, and ground storage must mean of necessity that evaporation and transpiration, or at least the algebraic sum of these two factors, are decreased.

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[45] Cons. Engr., Los Angeles, Calif.

The observations in the two localities show the following results for the second period (depths of run-off over the water-shed being given in inches):

	Wagonwheel Gap, Area *B*	Fish Creek
Actual	7.261	6.95
Normal	6.306	5.40
Difference	0.955	1.55
Percentage increase over normal	15.2	28.7

Transpiration by forest or brush cover certainly is greater than 1.0 to 1.5 in.—the increase in stream flow due to deforestation—and it may reasonably be concluded that a considerable portion of the decrease in transpiration following the removal of trees and brush is counterbalanced by increased evaporation from a warmer soil.

The fact that the two localities at which the observations were made had widely differing climatic characteristics makes the analysis of results of greater than ordinary interest. Wagonwheel Gap is at a high elevation, has a low mean annual temperature of 34° Fahr., with a range of temperature from below zero to 80° Fahr., and a relatively uniform seasonal distribution of precipitation, a considerable portion of it in the form of snow. Surficial run-off in Wagonwheel Gap is small, except when the snow is melting, most of the stream flow coming from ground storage, which, however, is receiving constant replenishment. The percentage of run-off to precipitation under normal conditions was from 29 to 30, which was increased after deforestation to 34.3. Fish Creek drains a much lower area, has a much higher mean annual temperature, probably about 60° Fahr., with, however, a smaller annual range (from below 32° to 100° Fahr. at Mount Wilson) of precipitation concentrated in a few months. Surficial run-off is large and little of the stream flow comes from ground storage.

An analysis of the increase of the actual run-off over the normal for both Wagonwheel Gap, Area *B,* and Fish Creek, for the period after deforestation, as presented in Table 17, is of interest.

The increase in actual over normal run-off appears to follow a different rule on each water-shed. At Wagonwheel Gap during November, December, January, and February, when the weather is very cold and snow is probably accumulating, the proportionate increase is small, increasing during the months of melting snow. Again, in June, there is an actual decrease, while in July and August the increase is small. The increase from November to May is probably due to faster melting of the snow, which, in turn, is caused by higher temperatures due to lack of shade from forest cover, this rate of melting being accentuated as the general temperature increases during March, April, and May. In June, soil evaporation is in excess of decreased transpiration, but during July and August it does not exceed the saving made through lack of transpiration.

On Fish Creek, the situation is different. Precipitation ceases in April or May, and stream flow is entirely supplied from ground storage from that time until November or December. The increase during the rainy season, due to increased surficial run-off which, in turn, is due to lessened ground absorption, is small. Ground storage, which has accumulated during the rainy season, and which before deforestation supplied the brush cover and was transpired, now appears as stream flow.

TABLE 17.—ANALYSIS OF INCREASE IN ACTUAL RUN-OFF OVER NORMAL RUN-OFF AFTER DEFORESTATION

Month	WAGONWHEEL GAP			Percentage of increase	FISH CREEK			Percentage of increase
	Actual run-off, in inches	Normal run-off, in inches	Difference, in inches		Actual run-off, in inches	Normal run-off, in inches	Difference, in inches	
October.......	0.363	0.329	0.034	10.3	0.06	0.01	0.05	500
November....	0.342	0.326	0.016	4.9	0.16	0.08	0.08	100
December.....	0.328	0.316	0.012	3.8	0.28	0.17	0.11	64.7
January......	0.313	0.302	0.011	3.6	0.32	0.22	0.10	45.5
February.....	0.287	0.274	0.013	4.8	1.93	1.57	0.36	22.9
March........	0.387	0.317	0.070	22.1	0.92	0.72	0.20	27.8
April.........	0.699	0.494	0.205	41.6	2.52	2.15	0.37	17.2
May..........	2.699	2.140	0.559	26.1	0.40	0.34	0.06	17.6
June..........	0.791	0.852	—0.061	—7.2	0.19	0.11	0.08	72.7
July...........	0.400	0.375	0.025	6.7	0.08	0.02	0.06	300
August.......	0.335	0.306	0.029	9.5	0.05	0.01	0.04	400
September....	0.317	0.275	0.042	15.3	0.04	0.00	0.04	∞
Total.....	7.261	6.306	0.955	15.2	6.95	5.40	1.55	28.6

A number of other things of interest are noted in the Fish Creek observations. In Tables 6 and 7 the effect of development within a short time of new cover on the reduction of maximum daily discharge, maximum peak discharge, and ratio of peak to daily discharge, indicates that it is grass and small vegetal cover that reduce surficial run-off, rather than heavy cover. The large increase in summer flow immediately after the fire on Fish Creek is probably due to the fact that increased erosion filled up the canyon with detritus and afforded an increased opportunity for ground storage, and that such fill was either eroded away, or through increased vegetation the water in storage was depleted through transpiration.

The analyses of the Wagonwheel Gap observations and of the Fish Creek data, as given in the paper, are distinct contributions to engineering literature, but broad conclusions cannot be drawn from them until much more quantitative data are obtained from other areas with different climatic, geological, and topographic characteristics.

Southern California has a marine type of climate. Moisture-laden air moves inland from the ocean, is forced to rise when it strikes the Sierra Madre Mountains, and in cooling, loses its moisture in the form of precipitation. The lack of summer rains is due to the fact that sufficient cooling does not occur during that period. Deforestation might be of value on a large scale under such climatic characteristics, provided that flood flows were not increased to a damaging extent, as it would not affect the source of supply for precipitation.

In a section with a continental type of precipitation, a considerable part of the moisture supply for precipitation comes from local evaporation and transpiration. Assuming an area several thousand square miles in extent (which had this type of precipitation source), to be deforested, the net evaporation plus transpiration would undoubtedly be reduced. Since it is the moisture received from these sources that goes to make up at least a part of the precipitation, the latter would likewise be reduced with a consequent reduction in surficial run-off and ground storage until a new state of equilibrium is established. As a specific example, consider conditions in the Southeastern States, where precipitation ranges from 50 to 70 in., stream flow from 20 to 40 in., and losses, which are made up of evaporation and transpiration, are constant at about 30 in., with none of the precipitation, or little of it, in the form of snow.

Deforestation would increase surficial run-off immediately following precipitation and undoubtedly would decrease ground storage. Whether the net change in surficial run-off plus ground storage would be plus or minus has not been established quantitatively, but the probabilities are that it would be a minus change. According to the observations in question, the change in evaporation plus transpiration would be minus, and since the equation of disposition of precipitation must balance, a reduction in precipitation undoubtedly would ensue. Since a large proportion of the precipitation during the growing season in this section is derived from local sources of moisture, the loss would be concentrated during this period, and might have a serious effect upon agriculture.

The foregoing discussion is given with a full realization that it is not based on any quantitative background, but with the purpose of showing that the conclusions reached from the observations in question cannot be accepted as general under all topographic, climatic, and geological conditions until many more quantitative data, collected under a wide variety of differing conditions, are available for analysis.

The methods used by the authors in the analysis of these observations are sound, and the conclusions drawn, as far as they apply to the water-sheds in question, are without question correct.

Ralph A. Smead,[46] Assoc. M. Am. Soc. C. E. (by letter).—The value of the analysis offered by the authors should be evident to any one who has given thought to the question of forests and stream flow. The writer was engaged with the U. S. Geological Survey some years ago in an earlier study of this problem under much less favorable conditions, and is doubly appreciative of the unusual opportunities for study offered by the areas analyzed in this paper. In 1911 and 1912, the Geological Survey conducted an investigation of a group of small water-sheds in the White Mountains of New Hampshire. As the field data covered only a little more than a year it was impossible to compare forested with deforested conditions on a single water-shed, and evidence had to be drawn from stream flow and precipitation records made simultaneously on the various water-sheds. Some of these water-sheds were covered with virgin forests, some were deforested by cutting, and some were denuded by forest fires. One great difficulty arose from the presence of numerous varying elements, other than forest cover, that might affect stream flow. Such elements include steepness of slope, exposure to sun and wind, size of water-shed, nature of soil, method of forest removal (whether by cutting or by fire), length of time since removal of forest, amount and nature of precipitation, etc. Even to estimate the effect on stream flow of such elements, and to determine the effect of forest cover alone, was far from a simple problem.

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[46] Structural Engr., Los Angeles, Calif. Mr. Smead resigned on January 9, 1933.

In the case of the Wagonwheel Gap and Southern California areas, on the other hand, such varying elements either were not present or could be eliminated effectively, due to the fact that a single water-shed could be studied both with and without forest cover, and due also to the presence of a closely similar neighboring water-shed by means of which the variations of meteorological conditions from year to year could be taken into account in determining the "normal" discharges (as the authors term them) for the period after forest removal. Thus, from these unusually adaptable areas, data have been furnished, and an analysis has been made that presents convincing conclusions as to the actual effect of forests on stream flow.

The authors call attention to certain variations in the run-off relations between Areas *A* and *B* of the Wagonwheel Gap section, and take those variations into account, probably by as exact a method as is practicable, in computing the "normal" discharge from Area *B*. It would be of interest, however, to investigate the causes for those variations, with a view to a more accurate determination of the "normal" discharge. For instance, the lag of several days between the spring peak discharges of Areas *A* and *B* before deforestation is apparently due to the difference in exposure to sunshine on the two areas, and the consequent delay in snow melting on Area *B*. This is indicated by the difference in direction of the slopes, by the disappearance of the lag at the time of the rain storm of October 5, 1911, and by the decrease in the lag between these peaks after deforestation had given the sun better access to Area *B*. It is possible that the relationships between cloudiness (if records exist), and the variation in amount of the lag would further establish sun exposure as the cause of this lag. Cloudiness records, then, for the period subsequent to deforestation would make possible even more accurate determinations of the "normal" discharges for this period.

The writer appreciates the careful selection and orderly arrangement of data as presented in the various tabulations and charts in the paper. The authors have confined themselves to the essentials of the problem in the interests of clearness and simplicity. However, the writer believes that one more chart, similar to that of Fig. 7, but covering an entire year of Area *B* at Wagonwheel Gap, would be worth while. It would show clearly and at a glance, as only a chart can show, the distribution, for the entire year, of the increase in annual run-off. It would also show many other main features of the analysis.

George H. Cecil,[47] Esq. (by letter).—The subject of this paper is of vital importance to Southern California. Perhaps nowhere else in the world, and certainly nowhere else in the United States, is water of such vital importance. While it is necessary to life everywhere, in a semi-desert region it is essentially the determining factor in urban and industrial growth and agricultural development.

From the standpoint of the value of crop production, the Valley of Southern California is the richest agricultural area in the United States. Already, in some localities, agricultural development has progressed beyond the assured long-time average yield of the local water resources. Given a cheap and adequate supply, many more acres of land are susceptible of development.

The importation of water into Southern California is possible and, in the case of Los Angeles, with its supply furnished in part from the ranges of the Sierras, it is an accomplished fact. Further importations from the Colorado River are contemplated, and plans for furnishing this supply to the Metropolitan Area are well advanced. These imported waters, however, will be needed eventually for urban supply and, even if they were sufficiently bountiful to permit their use for agriculture, the cost is prohibitive for any except the most valuable lands.

The Federal Government, the State, counties, and many of the municipalities of the region combined expend annually well over $1 000 000 in the protection from fire of the mountain areas of the region. This vast sum is expended not for the purpose of protecting a commercial timber supply or, for the most part, even a growth of tree species. The cover is, in the main, chaparral—a word derived from the Spanish and signifying a growth of dwarf oak. Trees are found in the canyon bottoms, at the higher elevations, and scattered here and there along the ridges of the chaparral covered slopes. There is ample evidence that these tree species at some time in the past comprised a more appreciable part of the cover, but at the present time by far the greater part of these mountain water-sheds is covered with a heavy growth of brush species, so dense (except where recently destroyed by fire), as to be impenetrable except on foot, and even then with the greatest difficulty.

Protection is afforded the cover on these areas on the assumption that: (1) It is a conserver of water; (2) it controls erosion; (3) it retards flood run-off; and (4) recreational value would be impaired by its removal.

If the authors are correct in their conclusions, the agencies now expending such vast sums in protection might well pause and consider the justification of these expenditures. These conclusions are, that such protection is not justified from the standpoint of the first objective; that the second is of little importance; the third, while the relationship is admitted in part, is apparently not considered of sufficient importance to affect the conclusions; and the same is, by inference, the case with regard to recreation since, although this form of use is mentioned, the conclusion reached is that damage by fire does not interfere with the recreational use of an area. Certainly, if all the factors of the situation have been given proper consideration and the conclusions are correct, the Federal Government, State, and local agencies would not be justified in the expenditures they are now making.

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[47] Executive Secy., Conservation Assoc. of Los Angeles County, Los Angeles, Calif.

There are several factors, however, that are important to the water user in Southern California, that have been overlooked entirely, and others that have not been given their proper value in arriving at a conclusion.

The mountain areas of Southern California perform their highest function in the production of a usable water supply for the municipalities and agricultural areas dependent upon them. Their use for recreation may be considered as of secondary importance.

The prime requisite in water production is that the water must be usable. This factor is of greater importance than the quantity produced, and is vastly more important than a minor increase in the sustained summer flow. Probably 95% of the water used for domestic and industrial purposes, outside the City of Los Angeles itself, is pumped, as is also upward of 80% of that used for irrigation. The reasons for this are several. The mountain canyons are steep and sites suitable for reservoirs are limited in number and expensive of development, precluding the storage of water except for municipal use where large communities are concerned. Further, the entire mountain area is faulted so badly and broken into blocks that the number of sites at which dams may be constructed safely, is few.

The mountain ranges of Southern California are the result of an upthrust of the earth's crust. The same, and subsequent, movements that gave birth to these mountain ranges and broke them into blocks, caused that part of the region lying between the foot of the upthrust and the Pacific Ocean to drop, thereby creating vast basins which, in the course of the ages, have been filled with detrital matter brought down from the mountains. This coastal plain is further characterized by the existence of lesser faults, resulting in the creation of numerous minor basins. In a study of the underground water of the South Coastal Basin, the State Division of Water Resources recognized no less than thirty-one such underground basins or reservoirs. The replenishment of these underground reservoirs which, as indicated, furnish the water supply for the major part of both urban and agricultural development, is of paramount importance.

In order that the water finding its way from the mountain areas on to the coastal plain may perform its maximum of use, as much of it as possible should percolate into the underground strata near the mouths of the canyons from which it issues. For years past, several communities, represented by the water companies supplying them, have spread the flood waters over the detrital cones by means of lateral ditches, increasing the wetted area and materially increasing percolation over that obtaining under natural conditions. The experience of these companies has proved beyond a doubt that, in order that water may be spread successfully and the maximum of percolation secured, it must be free of suspended matter. It is often necessary, during the first run-off of the season, to by-pass to the ocean a varying part of the flood flow. In the case of a water-shed that has been run over by fire, the quantity that must be by-passed because of the silt load is many times as great as that under normal conditions. It is admitted by the authors that the flood flows for the first few years after a fire carry an added quantity of silt.

Since repeated burning will be necessary to secure the increased flow that theoretically will result from destruction of the cover, this added silt load may be expected to continue until all erodable material has been carried off. Under such conditions, the water that will be lost to the ocean instead of being percolated into the gravels is many times the quantity that could possibly result from the added summer flow indicated. In fact, careful analysis of the estimated increase in the summer flow will show that although in percentage it reaches a startling figure, when reduced to actual run-off, in second-feet, it is almost negligible. Further, if a proper allowance is made for the solid material that will be carried at flood stage, and to some extent continuously, the increase in run-off for the winter or flood season will be decreased materially.

The quantity of eroded material carried by the floods from Fish Canyon is given little weight by the authors in arriving at their conclusions. The truth is that the detrital matter carried by Fish Creek, Sawpit Canyon, and Rogers Canyon, as an aftermath of the 1924 fire, not only damaged and destroyed a considerable area of agricultural land, but cost thousands of dollars for removal from highways and the railroad along the base of the mountains.

No flood-control dams have been constructed in these minor canyons. However, the Los Angeles County Flood Control District is spending approximately $25 000 000 in the construction of dams in other and more improtant water-sheds. The importance of water-shed cover to these engineering projects is freely recognized by engineers of the District. In a report on check dams dated May 22, 1931, its Chief Engineer, E. C. Eaton, M. Am. Soc. C. E., gives as one of his conclusions the following: "The most effective agent in both the regulation of débris and the reduction of flood peaks is the vegetation covering the watershed." Again, in a paper entitled "The Fire Problem and Erosion Prevention in Relation to Flood Control—Los Angeles County Flood Control District," presented at the Rural Fire Institute in February, 1931, Mr. Eaton states:

> "I have no figures on what fire prevention costs per year per square mile of watershed, but damages from débris deposits after fires, followed by only moderate storms, commonly run from $25 000 to $50 000 per square mile, and protective measures run anywhere from $2 000 to $20 000 per square mile. It is certain that prevention is much the cheaper."

In fact, the deductions made by Messrs. Hoyt and Troxell, if carried to their logical conclusion, would defeat the objectives sought in increased summer flow. With the destruction of the water-shed cover, the loose material would be carried eventually by the torrential rains down the steep canyons on to the valleys below. Even if the resulting damage could be ignored, as is suggested, the materials in which must be stored the water to produce the increased summer flow indicated, would themselves be carried away eventually. The result would be that the entire winter precipitation would immediately find its way to the valleys, and summer flow would be nil.

C. W. SOPP,[48] ASSOC. M. AM. SOC. C. E. (by letter).—In this paper the authors discuss a question that is becoming of more and more importance, due to the desire of certain persons to increase the appropriations for forest protection, of others to reduce such appropriations as an economy measure, and also the belief of certain men interested in grazing lands that periodic burning of the water-shed cover is beneficial to forage growth. All these matters vitally interest public officials and engineers who have charge of procuring and distributing a supply of water for domestic use and irrigation.

The authors' deductions that the removal of the forest cover increases the total run-off is fundamentally correct. However, it is that part of the run-off rendered usable that interests engineers and water company officials. Measurements of run-off at the mouth of the canyon in some cases are not a true index of the water supply, but rather the measurement of the water that is diverted or that replenishes ground-water basins by percolation. The maximum run-off would naturally occur from absolutely bare water-sheds typical of certain desert mountains, but this is not desirable when considered from all economic standpoints. Water supply officials are concerned with the maximum quantity of water rendered usable and not with the total run-off.

The increased flood run-off should be modified by a factor, dependent on the silt load, that will reduce the measured flow to water content. This is of considerable importance as even after three years of settlement of the first year's samples, the authors found that the silt was about 45% of the total volume. During the flood the bulking would be greater.

There is a direct loss of flood run-off after denudation by fire or by deforestation through the impossibility of diverting the silt-laden waters directly on the lands. On a denuded water-shed the flashy run-off occurs during or immediately after precipitation and at a time when the lands are saturated from the rainfall and there is no need for irrigation. The side-hill erosion deposits a large quantity of débris in the stream beds, which is sufficient in most cases to transform a rocky stream bed into a broad sandy one. In the course of a number of years all this deposition is moved out by the stream, causing great difficulties at head-works. The material is moved continuously in the form of material in traction, which rapidly fills all diversion works. This necessitates a waste of water in diversion either because it is possible only to skim the surface of the water or because of the necessity of flushing the settling basins. The sluicing and skimming losses may more than offset the gain in run-off. Great difficulty was experienced in attempting to divert even the clear water of San Dimas Creek, in Southern California, for a year or two after its partial denudation in 1919. In another case of diversion of water for domestic supply the material carried in traction into the head-works was 1 cu yd per hour per sec-ft of water. The capacity of the settling basin was such that the water had to be turned out frequently and the basin sluiced. For these reasons, in the absence of storage, there is actually a direct loss of water for irrigation and domestic use.

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Reservoir storage is an absolute necessity to make usable the increased run-off from a denuded drainage area, but erosion causes a rapid loss of storage space. One-half the drainage area above one reservoir in Southern California was burned over, and the run-off of the following winter season, even though it was a year of sub-normal rainfall, caused a loss of 11% in storage space. The deposition in the reservoir was 65% of the total run-off during the first year. This appears an abnormally high percentage, but it is not out of line with the percentage obtained by the authors during the first year, as shown by their table of silt samples (Table 12).

On another Southern California stream a flood-control reservoir was constructed the fourth year after a water-shed was denuded by fire. The subsequent deposition behind the dam has caused a considerable loss of storage and has interfered with the operation of the outlet gates. This illustrates that even four years in this case was not enough for the recovery of the water-shed cover to prevent considerable erosion. In Southern California, as a rule, five years are required for complete recovery under the most favorable situation as regards soil and moisture supply. After each fire the fertility of the soil is reduced by the burning of the humus and by erosion. The succeeding vegetation is of a lower type, and this process, if continued, will leave water-sheds practically bare.

There is an admitted increase in the summer flow from denuded water-sheds. In semi-arid localities the moisture in the water-shed exists as capillary water in the soil on the slopes and as free or gravity water in the alluvium in the canyon and valley bottom-lands. Under these conditions the removal of the forest cover from the side slopes would not cause an increase in the summer run-off inasmuch as the total supply available is only capillary water, and the removal of the transpiration loss would not increase the stream flow. The removal of the forest cover from the bottom-lands, especially from those water-loving plants and trees that have their roots in free water, does reduce the transpiration loss and causes an increase in stream flow. It has long been the observation of foresters that the summer or low-water flow increases on water-sheds after fires, but springs on the slopes do not so increase. This beneficial increase in water supply could be obtained by deforesting the bottom-land growth without any increase in erosion from the side slopes and probably very little increase in erosion of bottom-lands. The increased non-flood run-off is dependent on storage in the soil cover, but such storage would only be temporary because the continued practice of deforestation would eventually cause the removal of the soil storage.

In addition to the loss of water under conditions of direct diversion from denuded water-sheds and of the loss of valuable storage space in reservoirs, there is an indirect loss where an influent silt-laden stream traverses ground-water basins. The silt content tends to seal the stream beds and to reduce the percolation and recharge of these basins.

The ultimate result of continued denudation by fire or deforestation is to cause all run-off to occur in flashy floods when there is little demand for direct diversion for irrigation. It will also result in the erosion of soil cover that forms the reservoir from which the increased summer flow occurs, thereby removing the one beneficial influence derived from such denudation. Furthermore, it necessitates large and cheap reservoir space to make the increased run-off usable, and such storage is rapidly lost. The water-sheds are destroyed for all uses other than maximum run-off. Such denudation and deforestation should not be permitted, except where maximum total run-off is the highest use for which a water-shed has been classified, and cheap reservoir space is available.

W. P. Rowe,[49] Assoc. M. Am. Soc. C. E. (by letter).—The major portion of the mountain drainage area, tributary to the South Coastal Basin of Southern California, is covered with chaparral or brush. The true timber area comprises the lesser portion at higher levels. This area is used almost exclusively for recreational purposes and is visited each year by more people than any National Park in America. These two classes of vegetation are rarely found intermingled, probably because the rains are not sufficient to support both. Either will predominate to the exclusion of the other. No person has ever advocated the destruction of the true timber area in order to increase or control the water supply; therefore, this class will be omitted from the discussion to follow, the writer's remarks being confined to the chaparral areas of Southern California.

The Forest Service of the U. S. Department of Agriculture classifies chaparral as "forest cover," and this classification leads to confusion among engineers in the true timber localities, who are not familiar with the distinction between the two. Until the last few years, the Forest Service referred even to the most insignificant brush fire as a "forest fire." The local newspapers were of great assistance in spreading this misleading information. There is as much difference between burning the chaparral and timber as between paring a finger-nail and cutting off a finger, except that burning of the timber is soon offset by a rapid growth of chaparral, whereas a finger can never be replaced.

Chaparral growth follows a regular and never-ending cycle: First, comes the natural growth; then, a fire, started by any number of causes, burns off the chaparral. Then, follows the dire prediction of the professional conservationists that the water-shed is destroyed and the water supply ruined. The chaparral starts to sprout as soon as the fire is quenched. Then occur the winter rains and a little more soil is added to the valley floor and considerable more water is added to the underground basins that regulate 95% of the local water supply. The chaparral growth is back to normal in a few years, and the cycle is complete. These cycles have been occurring since the first lightning flash struck the dry chaparral, and they have occurred at more frequent intervals since the first Indian discovered that a chaparral fire resulted in more abundant feed for game and that it made hunting easier.

Every stream water-shed consists of a series of still smaller water-sheds each of which is traversed by a drainage channel. These drainage or stream channels accumulate the surplus water reaching the water-shed and convey it to the main streams. They are usually bordered by water-loving plants and act as natural fire-breaks. The steep side hills are quickly drained of surplus water and the end of each summer generally finds a veritable tinder-box awaiting the least spark to start a typical chaparral fire. There is a school of thought in California, composed principally of old-time settlers, *ex*-forest rangers, and hydraulic engineers versed in local water supplies, which advocates periodic and systematic burning of small areas of the chaparral. By burning the canyon ridges at alternate periods of ten years, there would be not more than 50% denudation in any small water-shed and there would be less than 10% denudation of a major chaparral-covered water-shed at any one time. The period of ten years is used only for the sake of comparison. It might be found that either a shorter or a longer interval was better.

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[49] Field Engr., San Bernardino; Engr., Coachella Val. County Water Dist., San Bernardino, Calif.

All the arguments that have been advanced against such a plan of controlled burning can be applied tenfold against the results of the present policy of protection. The fire that occurred in Ventura and Santa Barbara Counties in 1932 burned over entire water-sheds, and it is estimated that more than $100 000 was spent in trying to control it. A shift in the wind and accompanying fog extinguished this fire. It is estimated that more than $250 000 was spent in trying to control the San Gabriel fire in 1924, but a fog quenched this fire also.

The proposal of controlled burning of chaparral areas is not advanced as a measure for increasing the water supply, but as a simple method of fire protection to the recreational area and lessening, by at least 90%, the evils arising under the present system. The areas of most recent burning would be refuges for the game that is killed under the present system of maintaining areas of brush too thick and too extensive for quick evacuation in case of fire. The authors have exploded the theory that a burned-over area is destroyed as a water producer. It must not be overlooked, furthermore, that the very conditions that made it possible for the authors to draw their conclusions for chaparral-covered areas, were the result of a fire that did occur under the present policy.

W. C. LOWDERMILK,[50] ESQ. (by letter).—Questions are raised in this interesting paper concerning the influence of a mantle of forest vegetation, primarily on the yield of summer flow from water-sheds. The general conclusions, if tenable and if followed to logical ends, involve far-reaching consequences. The paper is one of the most important of recent contributions to the subject of forests and stream flow and requires scrutiny into methods used, analyses, and interpretation of data.

Conclusions are based on two water-shed studies, in diverse regions, under widely different climates. The Wagonwheel Gap area lies above an altitude of 9 000 ft in the snow belt of the Rocky Mountain region, within a continental climate in which annual precipitation is characterized by two approximately equal maxima, one as snow in winter, and the other as thunderstorms, or convectional storms, in summer. In this region the regimen of flow of the Rio Grande to which the Water-Sheds *A* and *B* are tributary, is well regulated, showing by channel conditions a minimum of sedimentation and irregularity of flow. The Southern California area, including Big Santa Anita and Fish Creek drainages, on the other hand, lies between elevations of 1 500 and 5 000 ft, within the coastal range of mountains, and within a typical Mediterranean climate in which precipitation is characterized by one maximum in winter and a long hot rainless summer. Regimens of stream flow show characteristic semi-arid type of irregularity with well developed outwash fans of coarse detrital material and extensive detrital filled basins. The Wagonwheel Gap study was conducted with great accuracy with respect to measurement of precipitation over water-sheds and environmental climatic factors and stream flow, whereas the Southern California study was an incident in stream gauging over a period of years along the mountain front of Southern California. No provision was made for an accurate determination of catch of water-sheds in question, nor of other climatic factors. Precipitation records of Mt. Wilson, an eminence on the west margin of the Big Santa Anita drainage area, are introduced with the assumption that precipitation fell at uniform ratios over the two drainages under observation. While the use of the unburned Big Santa Anita Water-Shed as an integrator of climatic factors is sound, it can serve legitimately only when total catch in precipitation is determined by measurements for the two drainages. Variation in rainfall from place to place within cyclonic disturbances in Southern California as shown in Table 18 is sufficient to impose a serious limitation upon the validity of the conclusions drawn from the study. Compare, for example, Items 8 and 9, using the catch of Big Santa Anita (Column (4)) as starting points. It will be noted that variations of 3 to 5 in. occur, in opposite directions.

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TABLE 18.—ANNUAL PRECIPITATION OF SELECTED STATIONS, SHOWING IRREGULAR VARIATIONS OF CATCH, SOUTHERN CALIFORNIA

Item (1)	Hydrologic years * (2)	Mt. Wilson (3)	Big Santa Anita (4)	San Gabriel Intake (5)	Pasadena (6)	Monrovia (7)	San Antonio (8)	San Antonio Intake (9)
1..........	1918–1919	21.98	.....	17.74	14.60	16.09	19.99	19.20
2..........	1919–1920	27.17	.....	25.55	16.77	19.69	30.43	27.98
3..........	1920–1921	34.28	33.95	26.30	19.59	24.41	34.58	32.90
4..........	1921–1922	60.26	57.33	57.58	29.36	31.73	55.35	54.18
5..........	1922–1923	24.48	26.71	23.60	14.01	17.16	27.02	27.66
6..........	1923–1924	16.76	16.05	14.20	8.94	11.87	19.46	19.76
7..........	1924–1925	21.91	21.15	16.95	12.88	16.34	19.89	19.36
8..........	1925–1926	36.26	39.60	31.42	22.44	22.21	37.01	37.34
9..........	1926–1927	33.37	39.17	34.19	25.13	27.24	38.67	38.51
10..........	1927–1928	19.78	18.46	15.26	13.73	12.84	18.40	20.41
11..........	1928–1929	20.68	23.03	23.10	16.42	17.57	28.10	.....
12..........	1929–1930	23.00	22.68	.....	15.79	18.33	27.19	.....
13..........	Average....	28.52	29.81	25.99	17.78	19.03	29.67	29.72

* Water-year, or hydrologic year, October 1 to September 30.

The authors' description of vegetation for the Southern California area is inadequate in comparison with that of the Wagonwheel Gap study. The inadequacy of the description of vegetative cover is apparent in references to the paper compared with Fig. 13 and Table 19, in which data collected from twenty-five sample plots, under the direction of Mr. A. E. Wieslander, for the State Type Map, were generously supplied, and are summarized. The vegetative survey was made in 1928, four years following the burning of Fish Canyon. Dominant vegetation was mapped in sample plots shown on the map, and this accounts for absence of reference to subordinate occurrence of yucca and other species named by the authors. Riparian, or canyon-bottom, vegetation is not included in Table 19. Field sample plots are indicated by numbered circles in Fig. 13. Type lines were drawn on base U. S. Geological Survey quadrangle sheets from occupied points, checked by detailed sampling in plots.

TABLE 19.—COMPOSITION OF CHAPARRAL VEGETATION IN FISH AND BIG SANTA ANITA CANYONS

VEGETATION		COVERAGE (AVERAGE PERCENTAGE)*	
Common name	Botanical name	Big Santa Anita drainage	Fish Creek drainage
Chamise	*Adenostoma fasciculatum*	20.00	10.0
Manzanita	*Arctostaphylos canescens*	4.5	6.0+
California sage	*Artemisia californica*	Trace	....
Whitebark soapbloom	*Ceanothus divaricatus*	8.0	7.3+
White hornbrush	*Ceanothus crassifolius*	13.0	1.2
Ceanothus	*Ceanothus oliganthus*	4.8	3.7
Birchleaf Mt. Mahogany	*Cercocarpus betuloides*	7.6	4.0
Wild buckwheat	*Eriogonum fasciculatum*	0.9	2.4
Grass	Grass	Trace	2.0
Christmas berry	*Photinia arbutifolia*	4.5	3.5
Hollyleaf cherry	*Prunus ilicifolia*	.....	4.6
California scrub oak	*Quercus dumosa*	8.0	11.0
Scrub interior live oak	*Quercus wislizenii var. frutescens*	8.0	5.6
Scrub canyon live oak	*Quercus chrysolepis nana*	.....	0.6
Deerweed	*Lotus scoparius*	.....	3.7
Black sage	*Salvia mellifera*	4.0	2.0
White sage	*Salvia apiana*	2.5	3.6
Red-berry	*Rhamnus crocea var. ilicifolia*	2.8	1.4
Sugar bush	*Rhus ovata*	4.0	5.5
Laurel sumac	*Rhus laurina*	1.5	....
Miscellaneous species		1.4	4.9
Bare surfaces		4.5	17.0
Litter, depth		0.5 to 3 in.	Trace
Total		100.0 unburned	100.0 burned over 1924

* Coverage refers to dominant vegetation; subordinate species under dominant cover are not listed.

Apart from the description of vegetation on slopes, the paper by Messrs. Hoyt and Troxell is entirely misleading with respect to the condition of the canyon-bottom growth of alders, willows, sycamore, poplar, and oaks of the woodland type. It is stated by the authors that this growth was protected from the fire and so it was below the falls in Fish Canyon. Above the falls, however, the mesophytic sub-irrigated canyon-bottom growth was entirely killed, as checked by the writer and recorded by numerous photographs. This discrepancy of fact and description imposes another important limitation upon the conclusions regarding summer flow which will be referred to again.

Absence of treatment of geological structure and soil formation imposes another limitation upon the study as a corroboration of the findings in the

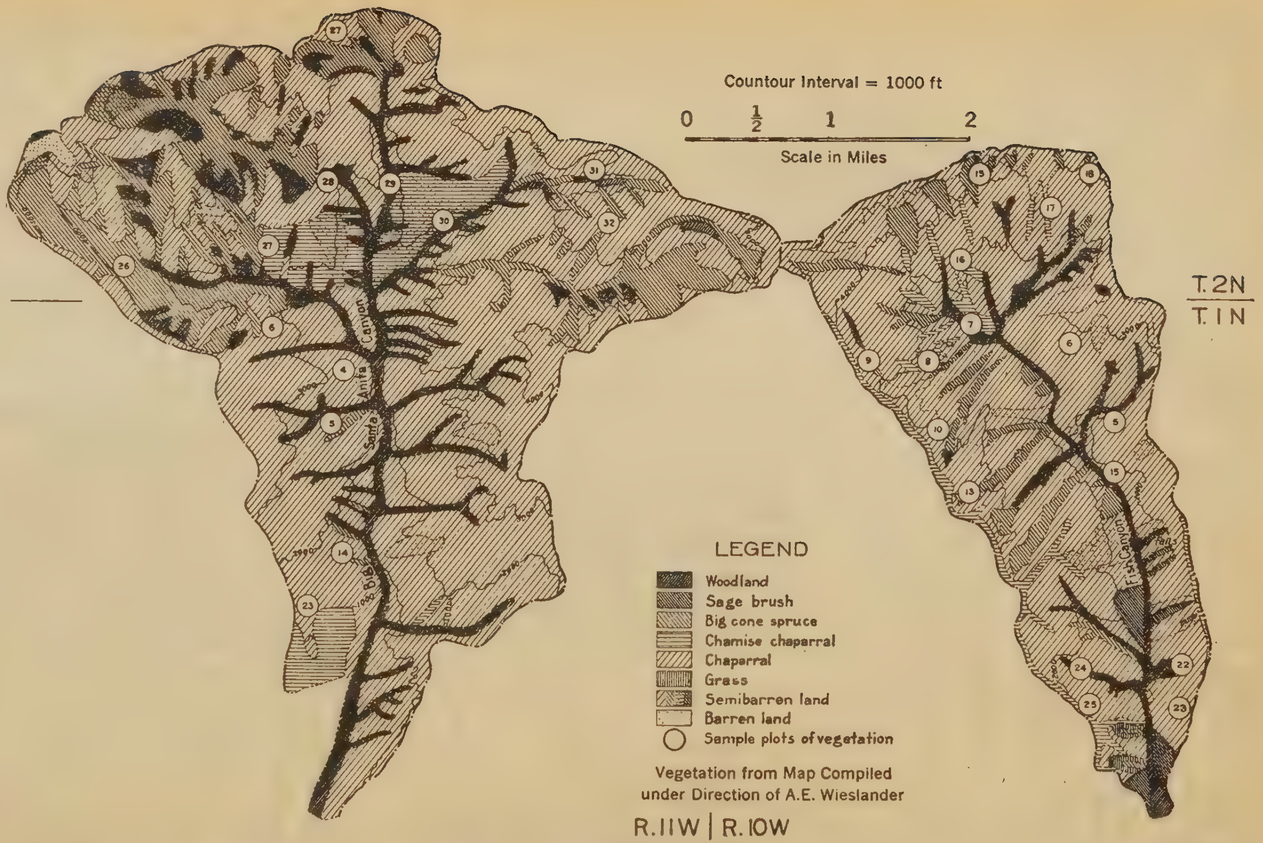
Countour Interval = 1000 ft
0 ½ 1 2
Scale in Miles
T.2N
T.1N
LEGEND
▩ Woodland
▩ Sage brush
▨ Big cone spruce
▤ Chamise chaparral
▨ Chaparral
▥ Grass
▨ Semibarren land
☐ Barren land
○ Sample plots of vegetation
Vegetation from Map Compiled under Direction of A.E. Wieslander
R.11W | R.10W
Big Santa Anita Canyon
Fish Canyon
27
28
29
30
31
32
26
27
6
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FIG. 13.—BIG SANTA ANITA AND FISH CREEK DRAINAGES, SHOWING COVERAGE BY VEGETATIVE TYPES AND LOCATION OF SAMPLE PLOTS OF 100 MILACRES, OR 0.1 ACRE EACH.

Wagonwheel Gap. Plotting of run-off against rainfall shows closer agreement between the catches measured in Big Santa Anita Canyon than at Mount Wilson, with a further indication that Big Santa Anita displays greater storage than Fish Creek under heavier intensity of rain when both areas were burned. The thin soils on the slopes of young mountains in Southern California offer comparatively small storage in comparison with deep soils of more gentle topography.

Before examining the principal question raised in this paper regarding effect of denudation on summer flow, it may be pointed out that the paper makes two important contributions: (1) It refutes certain important conclusions[51] of the late H. M. Chittenden, M. Am. Soc. C. E., cited in the paper; and (2) it indicates that erosion following denudation in Southern California is increased to a marked degree, to which experience and experiment fully attest.

A reference to the baneful influence of Colonel Chittenden's memorable paper is in place here. His conclusion that the "bed of humus" or litter of a forest floor is the chief agent of forest cover in absorption of precipitation is "an outstanding fallacy." Refutations of his conclusions in the discussion appended to his paper, require as much attention as his original paper. More recently the writer's studies[52] have shown that the capacity of a layer of forest litter to absorb water is of small importance in comparison with its function to maintain surface waters clear and the absorption capacity of underlying soil at its maximum. The mantle of soil in the final analysis is the absorbent of precipitation, not the forest. The forest cover, especially the litter which it produces, furnishes conditions which make for maximum possible water and erosion control on water-shed surfaces. It is curious that Colonel Chittenden's paper should continue to have its influence upon textbooks, writings, and opinions in this field of forests and stream flow when the late George Fillmore Swain, Past-President and Hon. M. Am. Soc. C. E.,

TABLE 20.—COMPARISON OF RUN-OFF FROM FISH CREEK DRAINAGE AS MEAN DAILY DISCHARGE AND AS SUMMER FLOW AFTER BURNING.

(Computations on the basis of the authors' data)

Hydrologic year	ANNUAL DISCHARGE, IN INCHES OF DEPTH			SUMMER FLOW: FISH CREEK			
	Big Santa Anita Creek	FISH CREEK		Normal, in inches of depth	Percentage of annual	Actual, in inches of depth	Percentage of annual
		Normal	Actual				
1924–1925	1.24	1.07	3.54	0.01	0.90	0.15	4.24
1925–1926	7.77	11.80	14.92	0.05	0.40	0.33	2.21
1926–1927	9.24	13.07	14.65	0.14	1.07	0.46	3.14
1927–1928	1.79	1.79	2.47	0.00	....	0.10	4.05
1928–1929	2.17	2.32	3.01	0.00	....	0.04	1.32
1929–1930	2.26	2.35	3.09	0.01	0.43	0.04	1.29

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[51] "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers", by H. M. Chittenden, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 245.

[52] "Influence of Forest Litter on Run-Off, Percolation, and Erosion," by W. C. Lowdermilk, *Journal of Forestry*, Vol. XXVIII, No. 4, April, 1930.

has presented[53] a much more open-minded statement of the problem and deserves to rank in authority much higher than Colonel Chittenden.

The point of particular moment in the paper by Messrs. Hoyt and Troxell is the effect of deforestation by cutting in the Wagonwheel Gap and the denudation by fire in Fish Creek on summer flow, which is commonly said to be reduced by deforestation. The relative flow is small in comparison with the total annual stream discharge as may be seen in Table 20. The ratio of summer flow and the percentages of increase are within the accuracy of measurement of stream flow in the study in Southern California. The contribution becomes not of so much importance in indicating substantial increase in yield of water as in the new and conflicting information it presents concerning a widely accepted belief. From a study of the two areas it appears that the indicated increase in summer flow occurs from two different causes. In the case of Wagonwheel Gap the difference is really of minor importance but can be accounted for by difference in interception of summer rain. Interception of as much as 40% of rainfall has been measured. From 10 to 20% is a conservative figure for a dense high forest cover. Since the increase in summer flow following deforestation in Water-Shed *B* was 9%, this amount may be accounted for by reduced interception of summer rains.

Such a possibility is eliminated in the case of the Southern California study in the absence of summer rainfall. Quite another explanation is required. In another paper[54] the writer has set forth an analysis of factors affecting the yield of water from Southern California water-sheds. The salient points are that the mantle of vegetation subdivides into two distinct types as may be noted in Fig. 13, namely vegetation of drained slopes (chaparral) and vegetation of sub-irrigated canyon bottoms (riparian vegetation). Chaparral vegetation on drained slopes has at its disposal after the end of the rainy season only that supply of moisture retained by the soil of the root zone against the pull of gravity, namely, available moisture between water-holding capacity and wilting point. A fire in summer will not release water from such drained soils. In the succeeding spring re-growth of sprouting vegetation and the great coverage of volunteer annuals may be counted on to extract moisture during the long dry season approximately to the wilting point within the soil occupied by roots of sprouting plants and invaded by new growth. In the event that root zones are not as fully occupied the first season following a fire as prior to a fire an increase of run-off coefficients (other factors remaining identical), may be expected. Such increases, however, under the climatic conditions of Southern California are not to be counted on to an important extent.

Quite another situation obtains with the riparian or canyon-bottom vegetation. It is typically mesophytic vegetation, and has at its disposal during the long hot summer an abundant water supply in canyons of perennial flow, such as Fish Canyon. Large quantities of water are transpired into the air by this vegetation. Accordingly, when a fire kills this riparian vegetation transpiration draft is stopped and increased flow in the stream occurs. Not until the riparian vegetation has returned to its former transpiration capacity will the flow be returned to normal.

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[53] "Conservation of Water by Storage", by George Fillmore Swain, Past-President and Hon. M. Am. Soc. C. E., Chester S. Lyman Lecture Series, Yale Univ. Press, 1914.

[54] "Studies of Factors Affecting Yield of Water from Watersheds in Southern California", by W. C. Lowdermilk, Proceedings Conference: Problems in Consumptive Use of Water and Conservation of Rainfall, Water Conservation Committee, Irrig. Div., Am. Soc. C. E., Los Angeles, 1930.

Since all riparian vegetation in Fish Creek above the falls (see Fig. 13) was killed in the fire of 1924, it is possible to account for the recorded increase of flow by reduction in transpiration losses of this type of vegetation along about 10 lin miles of main canyon and tributaries. For this reason the use of the Wagonwheel Gap experiment to support the findings from Southern California are brought into question as a scientific contribution.

This paper emphasizes the need of a thoroughly executed study in Southern California to answer satisfactorily the question raised as to method, and as to instrumentation and completeness in study of all factors. It must also be recognized that the question of transpiration loss on yield will not be the only basis for management of Southern California water-sheds for maximum beneficial yield of water. Storage in underground detrital-filled basins, control of flood discharges, and protection of flood-control reservoirs from sedimentation must enter into enlightened management of water-shed areas in Southern California.

RHODES E. RULE,[55] ASSOC. M. AM. SOC. C. E. (by letter).—The most important, and perhaps least understood, factor in the relation existing between vegetal cover and water supply is soil moisture storage. An idea prevails that significant quantities of moisture stored in the soil mantle during the rainy season are released later by slow drainage to surface streams, thus supporting the low-water flow of these streams. It is conceived that a dense brush or forest cover assists this process and thus acts as a conserver of water. Such misconception is due largely to mistaken ideas regarding the extent and velocity of soil moisture movement.

Many long-held concepts of soil moisture habit have fallen under the searching criticism of recent research workers. It is now definitely known that, in the absence of direct capillary connection with the water-table, movement of soil moisture under the influence of film forces is extremely slow. Other forces, probably closely related to the colloidal properties of a soil, tend to resist moisture movement and, as the moisture content of a soil decreases, the resultant force causing moisture migration becomes smaller and approaches zero at what Shaw[56] has termed the "normal moisture capacity" of a soil. At this point of static equilibrium the forces of adhesion and colloidal adsorption are great enough to prevent further movement and subsequent change of position must take place in the vapor phase. A wide range of moisture between this point and the hygroscopic coefficient, hitherto considered to be free to move under capillary forces, must remain in the soil until removed by evaporation or plant transpiration. Slow drainage is impossible after moisture in a soil has been reduced to this point.

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[55] Civ. Engr. (Rule & Rule), Los Angeles, Calif.

[56] "The Normal Moisture Capacity of Soils," by C. F. Shaw, *Soil Science*, Vol. 23, p. 303.

In Southern California the native plant life draws heavily on the soil moisture supply during the long dry summers and at the beginning of the rainy season there is a considerable deficiency of moisture in the root zone. Many determinations have shown that the entire root zone is close to the wilting percentage of soil moisture before the first autumn rains. The soil moisture reservoir is empty; it must be filled before general percolation can penetrate below the root zone.

The first rain will penetrate the surface layers of soil and quickly fill them to field capacity to a depth dependent on the soil moisture deficiency, the lower portion of the root zone remaining dry. Immediately, evaporation and plant transpiration activities commence and draw on this accretion of moisture. There is little opportunity for slow capillary adjustment to take place within the root zone. In a relatively short time evapo-transpiration losses reduce the soil moisture content to the "normal moisture capacity," and capillary moisture movement ceases.

Succeeding rains, provided they occur soon enough and in sufficient magnitude, gradually push the moistened area downward, but in the period between rains evaporation and transpiration continue to take their toll. If the total rainfall is heavy enough the root zone may finally be filled to field capacity and when this condition is reached deep percolation can begin. It can probably be safely assumed that most of the water that penetrates to below the root zone will be returned in time to surface streams as ground-water seepage.

However, water that does not escape the root zone during the first quick gravitational drainage period following wetting has a negligible chance of ever passing out of reach of the plant roots. After gravitational movement has ceased, the ensuing capillary adjustment is so exceedingly slow and plant demands are so great that escape is practically impossible in any appreciable quantity. Where, as is the case in vast areas of mountain water-shed, the root zone comprises the total depth of the soil mantle, storage within the mantle for later release to streams is impossible except in negligible quantities.

Certainly all ground storage, aside from the permanent ground-water body, which can be classed as possible of future capture by human agencies is of a deep-seated nature, that is, below the root zone. It consists of water in transit under conditions of excessive application. It is water in excess of equilibrium capillary capacity that is working its way down to the water-table. (Equilibrium capillary capacity is the point of soil-moisture equilibrium at which the forces of gravity and capillarity are in exact balance. It is the limiting condition approached in a soil which has been saturated from the ground surface to the water-table and then permitted to drain for an indefinite period without losses by evaporation or transpiration. Thus, it is the limiting moisture content of that part of the lithosphere lying between the lower horizon of the root zone and the water-table in a region periodically subject to deep penetration of precipitation to below the root zone. No water can reach the water-table until the moisture contents of all higher levels of soil or rock along the course of percolation have been raised to equilibrium capillary capacity.)

Where the water-table is in bed-rock, this storage is the water contained in the soil below the root zone in excess of equilibrium capillary capacity, plus the water contained in rock joints in excess of the equilibrium capillary capacity of the joints. The writer's observations of mountain springs and in deep tunnels lead to the conclusion that in mountain water-sheds deep-seated open joints are the principal transmission channels for waters that have escaped the root zone. This is contrary to the opinion sometimes expressed that the moisture that eventually feeds surface streams must percolate along or above the surface of the bed-rock, thus being continually exposed to capture by tree and brush roots which extend to bed-rock. If it is true that most of the waters of deep percolation in mountain areas are to be found in open joints, it is apparent that cover conditions which serve to preserve the soil mantle have no effect on maintaining a storage reservoir for such waters. On the contrary, the soil mantle may serve to hinder rapid passage of the moisture into the open joints and, by intercepting moisture and retaining it for plant use, may decrease the total deep percolation.

Thus, assuming the soils to be the same, compare the effect of a deep mantle *versus* a shallow one. If the root zone extends to bed-rock in both cases, the root zone storage (field capacity) of a 10-ft soil mantle will be twice that of a 5-ft soil mantle, and it will require twice as much water to fill the 10-ft soil column if the moisture deficiencies of the two columns are proportional to their depths. In this case deep percolation would start sooner in the 5-ft soil mantle than in the 10-ft mantle. It will be found, in general, that the deep soil mantle will support a more luxuriant cover, due to the greater moisture storage ability of its deeper soil column. This greater storage ability for the benefit of wild growth causes a reduction in the residual quantity available for capture by Man.

Hydrological studies of valley and foothill sections in Southern California have led the writer to the following opinions:

(1) General deep percolation in these sections occurs only in years of exceptional rainfall. It occurs in spite of, rather than because of, the native cover.

(2) The native cover has adapted itself to its water supply and under usual conditions is able to convert nearly all moisture that penetrates the soil to its uses.

(3) If economically practical, an appreciable increment to ground-water storage can be made available by "debrushing" of waste lands on valley floors and outwash areas.

There remain to be explained certain water supplies that can not be accounted for by totaling the inflowing waters to closed basins. These additions to ground-water supply must result from the concentration of rain water in pools and streams and the resultant local deep percolation of rainfall as distinguished from general deep percolation.[57]

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[57] "Water Supply from Rainfall on Valley Floors", by A. L. Sonderegger, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1242; see, also, discussion of this paper, p. 1269 *et seq.*

The observations on which the foregoing opinions are based, relate to valley and foothill sections of relatively small normal precipitation. Such sections are notably poor water yielders due to conditions of high evapo-transpiration losses relative to total precipitation. Data for the higher, more productive, water-shed areas are lacking. It is indicated, however, that consumptive use increases with an increase in precipitation. The evaporation opportunity becomes quantitatively greater and native cover more dense as precipitation increases. The statement previously made with reference to valley and foothill areas is probably likewise applicable here, namely, that deep percolation occurs in spite of, rather than because of, native cover. Much information is needed before definite concepts can be formed.

The only data available at present for studies of such productive water-shed areas are such as those presented by the authors in their timely and valuable paper. Resultant data of this nature are inadequate, however, for intelligent planning of water-shed control. The factors entering into the final results, as reflected by stream-flow measurements, are too many and too conflicting in their influences to permit final conclusions to be drawn as to how a particular water-shed or part of a water-shed should be managed.

In the final analysis the problem must be approached by hydrologic rather than hydrographic methods. The latter are valuable in checking the former, but are not sufficient in themselves. Evaluations of the separate factors of evapo-transpiration, deep percolation, soil moisture storage, and local surface run-off must be made for typical water-shed areas with characteristic cover types, before it will be possible to arrive at useful conclusions. Of course, these factorial evaluations must be supplemented by other studies of silting, damage to property, etc., but these considerations, although necessary, are apart from the present discussion.

In connection with the mention of the necessity for separate evaluations of consumptive use by different types of cover, the writer would like to have the authors discuss the subject of relative losses from the predominant chaparral cover of the Southern California water-sheds as distinguished from the water-loving growth found only in stream bottoms. From the writer's studies in a similar water-shed the possibility is suggested that the major portion of the increased summer flow after burning may have to be credited to reduction in consumptive use along the stream bottom, and that widespread burning of the chaparral cover may have had only slight effect on the low-water flow of the stream.

In conclusion, the writer wishes to urge that engineers and foresters, charged with the task of attacking the perplexing problems of water-shed management, make a thorough study of soil moisture behavior and the concepts of hydrology with respect to consumptive use of cover before committing themselves to definite policies.

ROBERT E. KENNEDY,[58] M. AM. SOC. C. E. (by letter).—Another rumor run to earth, one less alibi for procrustean argument! Heretofore, when it became necessary or desirable to explain more or less run-off, evaporation, or percolation as the case may be, it could be ascribed to the fact that Man had cut down the trees, drained the land, plowed up the sod, or done something to the water-shed. This paper contributes materially toward "nailing down" the bogy of deforestation.

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[58] State Engr., Bismarck, N. Dak.

Devils Lake, in North Dakota, has become known extensively as a classic example of the assumption that the same conditions which result from deforestation have operated to reduce its elevation about 16 ft since the first immigration to the water-shed in the Eighties. In this case, it was not deforestation but the changing of the tight prairie sod to open porous wheat soil. The result is said to be the occurrence of less run-off into the lake by reason of the increased loss due to more subsoil percolation, more plant consumption, and to crop exportation.

Much investment—Federal, State, and private—will be impaired if the lake finally dries up. One of the chief arguments for an enormous and fantastic engineering project to obtain water from the Missouri River, 100 miles to the west and 265 ft higher in elevation, was the assumption that the lake would not come back to any extent by natural causes, as other no-outlet lakes have done, because the introduction of the plow to the water-shed had upset the delicate balance in meteorological factors affecting it.

However, it seems evident that whatever other factors may prevail those conditions usually resulting from deforestation do not apply here because apparently they produce more run-off rather than less. Moreover, it will be difficult to prove that the mere changing of the soil cover from one type of sod to another has any appreciable effect.

The gauge heights do not show it. The lake fell 11.3 ft in the 20 years from 1867 to 1887. This is 6¾ in. per year. From 1887 to October, 1932, a period of 45 years, it fell 16.4 ft, or at the rate of 4⅓ in. per year. During the first twenty years of the record, only climatic conditions could affect it. If human agencies have modified it since, the evidence must be sought elsewhere.

A readily computable factor frequently quoted is the quantity of water shipped out in farm products; but that is negligible. Eighteen inches of rain each year is the equivalent to 2 000 tons to the acre. Fifteen bushels of wheat to an acre contains only about 130 lb of water; but according to the report of the State Commissioner of Agriculture only about one-half the land was plowed in 1928 and 1929. This would mean that not more than 75 lb of water per acre existed in grain products and that not all of it was exported.

It might even be asserted with some display of statistics that somewhat more run-off rather than less could result from the introduction of cultivated crops. Be that as it may, there is probably little run-off at any time. The terrain is a pan and pot-hole, knob and kettle type of pleistocene glacial drift lying on cretaceous shale of the Pierre formation,[59] concerning which Professor Howard Simpson, of the University of North Dakota, states, "the region is almost unmarked by drainage lines save a few shallow winding coulees through which little or no water flows to-day."

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[59] *Bulletin No. 598,* U. S. Geological Survey, p. 73.

There is a reported subsoil inflow of considerable but unmeasured quantity. If it has increased with the coming of the white man that development would tend to raise the lake rather than lower it; but there has been no report of a noticeable increase in the elevation of the ground-water table. No extensive land drainage projects have become necessary.

The lake has no doubt been dry or nearly so a number of times before. The same forces that filled it before are in all probability still operating without hindrance. It may be that, in the far distant future, great engineering works will be necessary to keep lake water out of the City of Devils Lake itself, now about 50 ft, or more, above the present water surface.

HERMAN STABLER,[60] M. AM. SOC. C. E. (by letter).—The American people are fond of hokum, and none is so prone to accept its dictates as scientists—seekers after truth—who delve in theories in the absence of substantial factual foundation. Perfect logic starting from false premises will lead to false conclusions. A consensus of scientific opinion in the United States thirty-five years ago on the subject of forests and stream flow is contained in the following excerpts from a report of a Committee (Messrs. Charles S. Sargent, Henry L. Abbott, A. Agassiz, William H. Brewer, Arnold Hague, Gifford Pinchot, and Wolcott Gibbs) of the National Academy of Sciences[61]:

> "The influence of forests upon climate, soil, and the flow of water in streams has attracted much attention during the past century; but while the general consensus of opinion among experts is that this influence is potent and beneficial, the absence of exact data extending over long periods of time, and the complex nature of the phenomena involved, render it necessary to base this conclusion rather upon general considerations than upon statistics."

With this introduction the Committee painted a picture of the natural regimen of streams "replaced by destructive floods in the spring, and by dry beds in the months when the irrigating flow is most needed" and of prosperous communities "depopulated" by reason of assumed destruction of forests on mountainous water-sheds of the arid regions of North America; declared that "unrestricted pasturing of sheep" must inevitably render "worthless for irrigation" streams rising in the Sierras and Southern Cascades; and concluded with the general opinion,

> "That, while forests probably do not increase the precipitation of moisture in any broad and general way, they are necessary to prevent destructive spring floods, and corresponding periods of low water in summer and autumn when the agriculture of a large part of Western North America is dependent upon irrigation."

It is little wonder that, with such backing of scientific opinion—even though based admittedly "rather upon general considerations than upon statistics," aided and abetted by the natural fondness of most people for trees and the undeniable value of forests as sources of wood and lumber—nationwide propaganda for forests as conservators of water and regulators of stream flow has taken firm hold on the imagination of the people and that it has become so firmly rooted in the popular mind as to be regarded as more or less axiomatic. Messrs. Hoyt and Troxell are to be congratulated for bringing before engineers statistical results in succinct form on the basis of which the volumes of propaganda, written and preached to the American public, may be adjudged. These statistical results, obtained in the very region chiefly considered by the Committee of the National Academy of Sciences, show that the Committee erred in assuming that loss of vegetative cover would produce dry beds of streams in summer and render them worthless for irrigation, and that claims heretofore made in behalf of forests on the basis of "general considerations" are clearly fallacious in some areas and, in the absence of statistical data to the contrary, must be assumed to be equally fallacious elsewhere.

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[60] Chf., Conservation Branch, U. S. Geological Survey, Washington, D. C.

[61] Report of the Committee Appointed by the National Academy of Sciences upon the Inauguration of a Forest Policy for the Forested Lands of the United States, to the Secretary of the Interior, May 1, 1897, Washington, Govt. Printing Office, 1897.

The writer would be particularly interested to see the results of an experiment, such as that conducted at Wagonwheel Gap, undertaken in a humid region and under conditions most favorable to maintenance or betterment of low-water flow by vegetation. Such an experiment would substantially determine whether the principles set forth by Messrs. Hoyt and Troxell are universally applicable. Until such an experiment, conducted under trustworthy auspices, proves the contrary, one must assume universal applicability of these principles because of the wide diversity of conditions covered by their studies.

No one can be more interested than the hydraulic engineer in the actual facts of the case, and it is hoped that engineers will assist in accumulating a mass of statistical data relative to effects of vegetation on stream flow, that will furnish conclusive evidence applicable to a great variety of conditions. Engineers, however, are not by nature missionaries or propagandists. It will remain for some other group to educate the public. Is it too much to hope that those who have been most active in spreading doctrines now found to be fallacious will be equally zealous in disseminating the results of trustworthy statistical studies?

As a supplement to the studies of Messrs. Hoyt and Troxell mention should be made of another attempt to determine the effect of vegetation on run-off and erosion. This study was conducted on grass- and weed-covered slopes of the Wasatch Mountains, in Utah, and is reported by Mr. C. L. Forsling.[62]

The published results of the experiment show wide discordances in precipitation, run-off, and erosion for different observations. Amount and distribution of rain, intensity of precipitation, condition of surface as to moisture, temperature, marked differences in topographic and channel characteristics of the two areas, etc., all serve to obscure whatever influence vegetative changes may have exercised.

After extensive independent study of all results of the experiment the writer has concluded that summer storms, which produce most of the destructive run-off and erosion in the Wasatch region, offer the most instructive field for comparison with the results found by Messrs. Hoyt and Troxell. He, therefore, presents herewith a brief description of the conditions of the experiment and of the results of his study of the data relating to important summer rainfall.

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[62] "A Study of the Influence of Herbaceous Plant Cover on Surface Run-Off and Soil Erosion in Relation to Grazing on the Wasatch Plateau, Utah," by C. L. Forsling, *Technical Bulletin No. 220,* U. S. Dept. of Agriculture, Washington, D. C., March, 1931.

Two adjacent areas of about 10 acres without perennial run-off were selected for study, Area *A* having a vegetative cover of 16% and Area *B*, a cover of 40 per cent. Collecting basins were constructed to catch all run-off and sediment, and rain-gauges were set up to measure precipitation amounting to about 30 in. per year. Observations were continuous from 1915 to 1929. Area *B* was maintained with 40% cover throughout the period by light grazing of cattle or sheep once or twice each summer. Area *A* had 16% cover through 1919, this being maintained by grazing as on Area *B*; increase to 40% was attained from 1920 to 1923 or 1924, by elimination of all grazing and the sowing of grass seed (wheatgrass and bromegrass) near the upper end of the area; and this higher value was maintained until the end of 1929, being kept down by grazing very lightly by cattle in September, 1925, and moderately by sheep once in late September or early October of 1927, 1928, and 1929.

TABLE 21.—DATA RELATING TO JULY AND AUGUST STORMS.

Date of storm	Average precipitation, in inches	RUN-OFF, IN INCHES		SEDIMENT, IN CUBIC FEET	
		Area *A*	Area *B*	Area *A*	Area *B*
August 2, 1916	0.34	0.0105	0.0064	15.9	5.1
August 5, 1916	0.26	0.0122	0.0121	8.1	4.8
August 9, 1917	1.04	0.0997	0.0156	67.2	6.4
July 9, 1918	0.27	0.0215	0.0002	26.6	0.2
July 10, 1918	0.21	0.0121	0.0002	6.1	0.2
July 13, 1918	0.72	0.0612	0.0032	60.1	2.0
July 12, 1919	0.58	0.0081	0.0004	6.9	Trace
August 8, 1920	0.84	0.1532	0.0248	79.0	28.5
August 25, 1920	0.44	0.0206	0.0074	17.1	7.7
July 15, 1921	0.62	0.0275	0.0009	11.1	0.6
July 17, 1921	1.25	0.2325	0.0534	82.7	23.1
August 15, 1921	0.46	0.0296	0.0016	12.3	0.6
August 23, 24, 1921	0.79	0.0405	0.0201	16.0	12.5
August 25, 1921	0.08	0.0043	0.0003	1.1	0.3
August 26, 1921	0.20	0.0171	0.0171	8.6	11.5
August 27, 1921	0.08	0.0001	0.0024	Trace	0.3
August 28, 30, 1921	0.56	0.0308	0.0273	19.4	11.5
August 1, 1922	0.70	0.0432	0.0059	23.7	2.9
August 31, 1922	0.36	0.0293	0.0128	23.6	18.5
August 13, 1923	0.86	0.1627	0.0572	75.1	25.1
July 27, 1924	0.84	0.0007	0.0007	Trace	Trace
July 4, 1925	0.20	0.0064	0.0053	1.1	Trace
July 3, 1926	0.48	0.0017	0.0011	Trace	Trace
July 4, 1926	0.36	0.0140	0.0093	11.7	2.4
July 21, 1927	0.70	0.3270	0.1326	55.5	30.0
July 27, 1927	0.56	0.0895	0.0735	28.9	13.2
July 28, 1927	0.22	0.0045	0.0029	0.8	0.3
August 5, 1929	0.63	0.0299	0.0017	14.7	0.5

Table 21 gives the statistical data as recorded by Forsling for July and August storms that produced run-off on both areas and that showed variation of not more than 25% in precipitation on the two areas.

By simple arithmetical operations, the progressive accumulations of precipitation, run-off, and sediment, the progressive accumulations of excess run-off and sediment from Area *A* over run-off and sediment on Area *B*, and the ratios (Area *A* to Area *B*) of progressive accumulated run-off, sediment, and sediment per inch of run-off, were determined from the data in Table 21 and are presented in Figs. 14, 15, and 16.

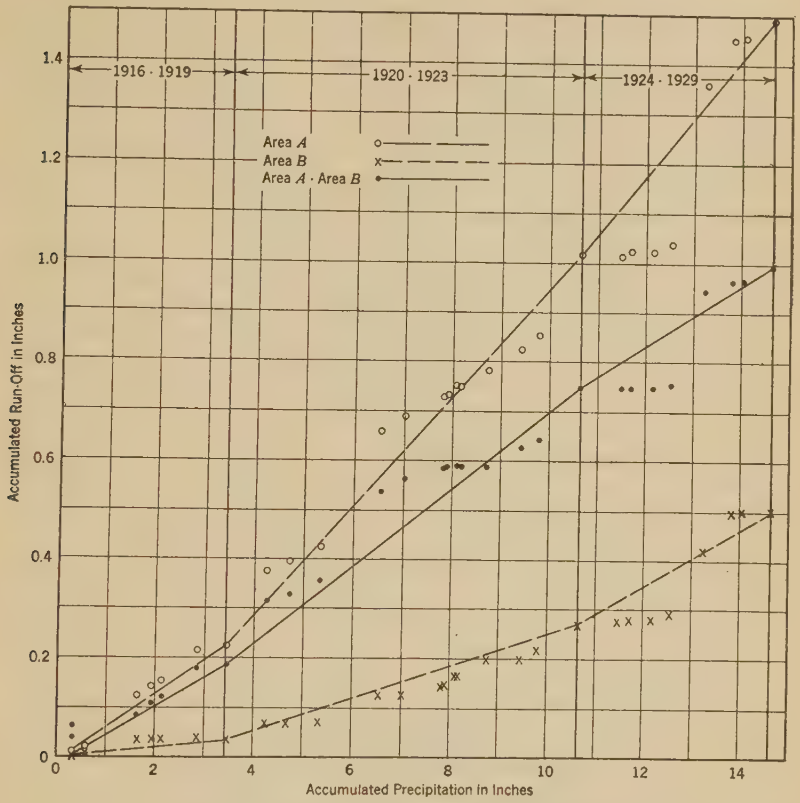

FIG. 14.—RUN-OFF—PRECIPITATION RELATIONS

Fig. 14 indicates that, relative to precipitation occurring in summer storms, Area *B* (vegetation constant) produced run-off at a higher rate in the second period (1920-1923) than in the first period (1916-1919), and at a still higher rate in the third period (1924-1929). Area *A* (vegetation variable) shows a similar trend, although the rate of run-off in the third period is very slightly greater than in the second. The excess of run-off from Area *A* over run-off from Area *B* shows very little change in rate. The inference to be drawn from these curves is that vegetational changes on Area *A* affected run-off but little—less, indeed, than other factors.

Fig. 15 indicates that, relative to precipitation occurring in summer storms, sediment was washed from Area *B* (vegetation constant) at a higher rate in the second and third periods than in the first, the rate being slightly less in the third period than in the second. Sediment was washed from Area *A* (vegetation variable) at practically the same rate in the first two periods, but at a slower rate in the third period. The excess of sediment washed from Area *A* over sediment washed from Area *B* was at a lower rate in the second period than in the first and at a lower rate in the third period than in the second. These trends strongly suggest that as vegetative cover on Area *A* was increased sediment removal was decreased. Doubt on this suggestion is cast by the fact that Area *B*, with constant vegetation, showed a greater rate of change than did Area *A* and that most of the decrease in excess of sediment (Area *A* over Area *B*) is due to the marked increase in sediment removal from Area *B* during the second and third periods.

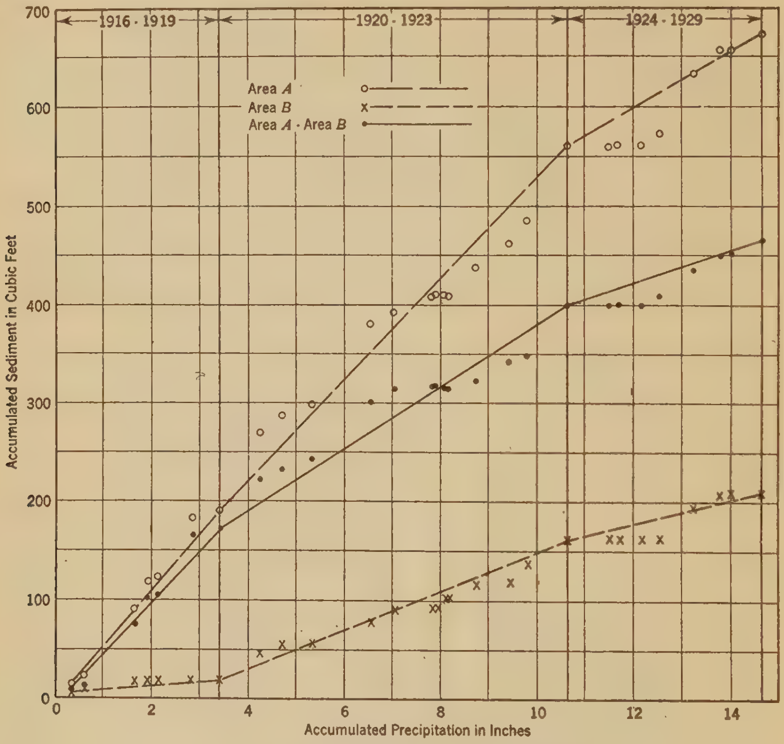

FIG. 15.—SEDIMENT—PRECIPITATION RELATIONS

Fig. 16, which deals only with relative results and, therefore, depends wholly on the reliability of Area *B* as a constant or "blank" for purposes of comparison, indicates, relative to precipitation occurring in summer storms, a striking reversal of trend both for ratios of run-off and for ratios of sediment removal. The ratio of sediment removal per inch of run-off, nearly constant at 1.7 during the first period, drops abruptly to another approximate constant, only about one-half as large, at the beginning of the second period, and changes but little from that point on to the end of the experiment. The third period shows, with respect to the second, a slight increase in the run-off ratio ($A:B$), a slight but somewhat greater increase in the sediment ratio ($A:B$), and a slight increase in the sediment ratio ($A:B$) per inch of run-off. These trends strongly suggest that elimination of grazing on Area *A* at the end of 1919 and the concurrent sowing of grass seed on the upper part of the area effected an immediate overwhelming change in the run-off and sediment trends, relative to Area *B*. Substantially the entire change seems to have occurred between July, 1919, and August, 1920. Yet Forsling's report states that "although grazing was discontinued in 1920, there was no change in the vegetation in that year." If this statement, surprising in view of the sowing of grass seed, can be accepted at face value one must look to the presence or absence of grazing animals rather than to any change in vegetation for the reversal of trends. This thought gains some strength from the observation that, in the third period, after grazing on Area *A* had been resumed in part, there is an indicated tendency toward return to conditions of the first period. However, bearing in mind that most of the trends are due to variations on the supposed constant Area *B* (which, as noted, is not a fixed quantity), all conclusions reached must be considered as tentative if not speculative.

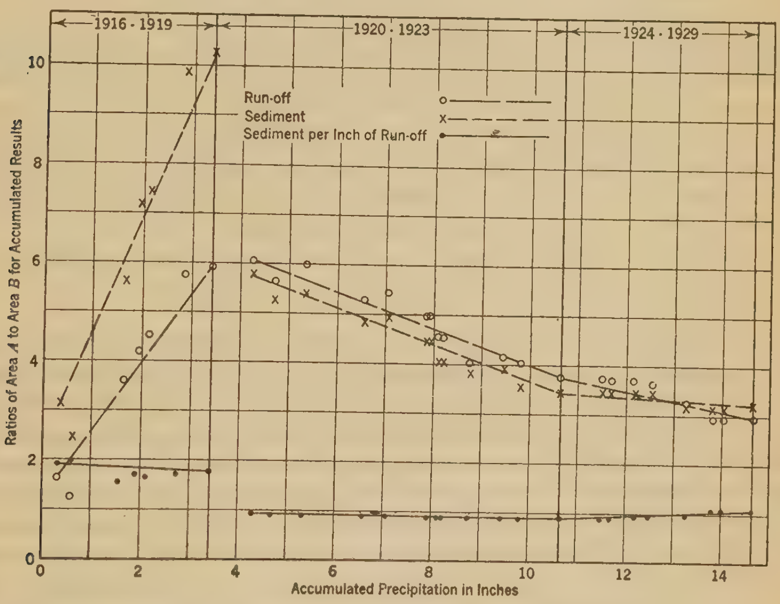

FIG. 16.—RELATION OF RATIOS TO PRECIPITATION

If the curves had been constructed with time elements instead of precipitation as abscissas, the slopes of all lines in the second period would be steeper and those in the third period flatter with reference to the slopes of lines in the first period. This follows directly from the record of 3.42 in. of rainfall in the four summers of the first period, 7.24 in. of precipitation in the four summers of the second period, and 3.99 in. of precipitation in the six summers of the third period.

The writer is inclined to draw the following tentative conclusions from his studies of the Wasatch Plateau experiment in the light of the study by Messrs. Hoyt and Troxell:

1.—On the steep grass and weed-covered mountain sides of the Wasatch Plateau, Utah, wide changes in vegetative cover and in grazing practices (*a*) produce little change in the rate of run-off from summer storms, slightly more run-off with more grazing and with less vegetation being indicated; and (*b*) affect notably the rate of sediment removal from summer storms, more erosion with more grazing and less vegetation being indicated.

2.—Under the conditions of soil, relief, and climate affecting the experimental areas, measures that will restore and maintain the maximum plant cover for grazing purposes will also insure adequate water-shed protection.

3.—A cover of grass and weeds, grazed under careful regulation, may afford protection to soil as satisfactorily and more profitably and deplete water supply less than a cover of useless brush. Relative effects of grass and weeds, brush, and trees on water supply and soil protection should be studied to afford a reliable guide in administration and use of lands.

4.—If wide variations in vegetative cover and grazing practices on steep mountain sides where the annual precipitation is about 30 in. produce changes in run-off and erosion no greater than indicated by the Wasatch Plateau experiment, neither variations artificially producible in the annual forage crop of about an ounce to the square yard on the relatively flat-lying and arid unreserved public domain (170 000 000 acres), nor the presence or absence of a stock population sufficient to harvest it, can affect appreciably problems of water supply or sedimentation of the arable lands of the West.

5.—The study, in general, tends to corroborate the conclusions reached by Messrs. Hoyt and Troxell and to extend their application.

H. S. GILMAN,[63] ESQ. (by letter).—It is quite possible that there are areas of water production and use, throughout the world, that will be greatly benefited by the findings and recommendations embodied in this paper. It is also possible that the authors' recommendations as to water-shed management will be adopted in field practice in an attempt to increase water yield. The result of actual field practice, if accompanied by close observations of all factors in water yield, may finally settle this old controversy of the beneficial or detrimental influence of forests upon stream flow and water yield. It is a surprising coincidence, however, that the observations upon which this discovery and findings were based, occurred on areas in Southern California, where, because of a combination of natural conditions, water-shed protection is popularly supported and practiced more intensively possibly than in any other area. Steep mountain slopes, a shattered rock formation, a layer of soil, a covering of scrub forest native to a semi-arid region, the year's rainfall occurring for the most part within a period of ninety days, few and costly reservoir facilities, a few miles of travel for the water from the mountains to the ocean—this briefly sets forth the water-producing machinery, and the authors recommend eliminating the vegetative cover and speeding up the run-off.

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[63] Civ. Engr.; Member, California State Board of Forestry, San Dimas, Calif.

As manager of a mutual water company in Southern California for the past seventeen years, the writer has been in charge of diversion, production, and distribution of water for irrigation and domestic use, in an area of water production and use very similar to the areas of Santa Anita and Fish Canyons compared in this paper.

As is the case in most parts of Southern California, the annual water crop is produced from two sources—diversion of surface flows of the mountain canyons, and pumping from the alluvial basins beneath the plains and valleys. The surface gravity flows are unreliable in quantity as a source of supply, due to the seasonal variations in rainfall. The basins are more reliable as a constant source of supply, and act as an equalizing reservoir to carry over the years of deficient rainfall. For the past seventeen years, 20% of the Company's supply has come from gravity water and 80% has been pumped. Therefore, basin water, although higher in cost of production, is the most valuable because of its greater dependability.

Many water companies and agencies, recognizing the value of underground storage, have expended large sums of money in artificial works to increase percolation of storm waters over the débris cones at the mouths of the canyons for the purpose of recharging the basins.

In 1919, a forest fire denuded 50% of the slopes of the water-shed tributary to the area from which the Company produces its water, and the writer has direct knowledge of the effect of cover disturbance upon actual water production as opposed to a theoretical study which touches on one phase only, that of surface flow.

Water conditions in Southern California are such that maximum recovery of the annual precipitation must be made in both diversion of stream flows and through the pumping of underground waters. Any practical solution or suggestion as to water-shed or basin management that will increase the yield will be quickly applied to the problem by those in charge of water production. Arguments as to the benefits of denudation are as old as the hills themselves. The paper contributes nothing new as far as authentic data are concerned; and it overlooks, or sets aside as being of no consequence, certain factors that control water production, in all its phases, in this area. It is simply another instance of a plausible theory that will not stand the test of field practice.

The application of Mount Wilson rainfall data as a measure of precipitation upon both the Santa Anita and Fish Canyon areas to develop a formula for comparison of actual as against theoretical flows, in this scientific discussion, can scarcely be accepted as being sufficiently authentic to place in effect so radical a change in water-shed management as is recommended. Mountain rainfall records in Southern California are few and far between. Such as are available indicate great variations in seasonal storms and hourly intensity in the various areas. Variations up to 100% have been noted between stations ½ mile apart for various storms, and the picture of uniform precipitation in quantity and intensity over the areas reported, can be based only upon a wild assumption.

The U. S. Geological Survey records of stream flow in Southern California are full of examples of great variations per square mile of water-shed run-off between adjacent or distant drainage areas, where no cover disturbances have occurred. An intensive observation of actual rainfall "catch" upon these areas under discussion might have indicated a greater or lesser increase in yield, but under the circumstances the findings are valueless.

Besides questioning the authenticity of rainfall data used by the authors, other fundamental objections may be taken in their attempt to show a beneficial increased yield in production by reasons of denudation.

In the first place, the measurement of water over a weir is not the final measurement of water yield. In practical diversion and application to use, the condition and usability of the water, as well as the relation of mountain precipitation and run-off to basin recharge, are as important as the quantity. In the paper, while reference was made to the great percentages of solids in the water, apparently no correction of weir measurements was made, and everything flowing over was counted as water to show the beneficial increase. In practice of diversion and use, however, it is necessary for those in charge to make such corrections, not on paper, but physical corrections in the field, before the water can be used for spreading purposes or delivered and sold for use. Suspended matter in water prevents percolation into the basins and is objectionable in both irrigation and domestic use. The writer found that before the 1919 fire the stream cleared within two or three days after a storm so that it was usable for any purpose. For four years after the fire, suspended matter in the water and the stream-bed flow of sand and gravel, upset the old program of diversion and use to such an extent that any benefit in increased yield as indicated by Messrs. Hoyt and Troxell (including as it does the solids), was more than offset by necessary loss of water through the sluicing of settling basins before diversion could be made. The small increase in summer flow would be a high price to pay for the loss of percolation of winter waters in recharging the more important underground supplies.

That the data used by the authors permitted findings parallel to those for Wagonwheel Gap can only be laid to accident. If reliable data had been available and if the findings resulted as reported, the recommendations would still not be practicable in Southern California in the absence of recommendations as to erosion control, desilting, and a safe and economical method of denudation.

Considering their findings that, where flood control is a major problem of a drainage area, the maintenance of an undisturbed cover is desirable, it is difficult to visualize an area where population and irrigation demands are such that an increased yield of water is so imperative as to demand the drastic treatment of a water-shed by deliberate denudation, at the same time exposing itself, its people, and values to an increased flood threat.

Several counties in Southern California are spending many millions of dollars in the construction of engineering projects in an attempt to control or retard the normal and flood flows during the winter seasons. These projects, for the most part, are storage reservoirs in the mountains, usually constructed at high cost per acre-foot of storage. These reservoirs are expected to operate under conditions of normal water-shed cover conditions. It is questionable whether they would function under a denudation program of water-shed management, in the light of the authors' showing of a great percentage of increased run-off during the period of normal heavy flow. Furthermore, the economic life of a high-cost reservoir should be a matter for consideration in the light of the great increase of eroded material in the flow. The great percentage of increased yield during a period of heavy normal yield under these conditions, raises the question as to whether or not the increase is beneficial, while the lesser increased yield during the period of normal flow, or no flow, amounts to a surprisingly small quantity of actual water when computed in dollars and cents.

It is the writer's opinion that men in charge of actual water production and diversion of water in Southern California are as strongly in favor of water-shed protection as ever.

Charles H. Lee,[64] M. Am. Soc. C. E. (by letter).—The authors have presented a thorough and convincing analysis of the effects of deforestation by cutting, and denudation by fire, respectively, upon the stream flow of two small mountain water-sheds. They have done well in distinguishing between deforestation by cutting, and denudation by fire, because the destruction of the forest litter in the latter case constitutes a fundamental difference when considering the effect upon stream flow.

Although much impressed by the conclusions respecting these two water-sheds, the writer does not believe that the same conclusions can be extended indiscriminately to other water-sheds. Justification for this statement appears when it is found that differing causes produced similar results in the two water-sheds under consideration. For example, the writer recently examined precipitation data carefully observed during a 4-year period at eight stations in an area of mixed coniferous and deciduous forest trees, which indicate an average interception of rainfall during the summer months of 22 per cent. This percentage would fully account for the 12% increase in summer stream flow from the Wagonwheel Gap area after deforestation, even allowing for errors due to leaf drainage, etc., since this flow is derived principally from copious summer storms (Table 2). At Fish Creek, on the other hand, summer rainfall is practically zero, and stream flow is derived entirely from ground-water storage. The cause for the 475% average increase in summer stream flow, amounting to 65 acre-ft per annum (Table 9), can here be entirely accounted for by the burning of riparian vegetation which extended along the stream channel for fully ten miles (see Fig. 13). In other words, similar changes in stream flow resulted from entirely different causes. Further search might disclose other instances of apparent but not real agreement in the results for the two water-sheds.

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The writer is firmly of the opinion that correct general conclusions cannot be reached by taking into consideration forest and stream-flow data alone, but that all the hydrologic elements which influence stream flow must also be considered. Most of these elements are generally recognized, and may be briefly summarized as follows:

Precipitation upon a water-shed finds disposal from the area either as water-shed losses, stream flow, or sub-surface leakage. Water-shed losses consist of (*a*) evaporation from leaves and branches by interception (immediate); (*b*) evaporation from the soil (within 10 to 18 days after a storm); (*c*) evaporation from snow on the ground (within 2 to 6 months after a storm); (*d*) transpiration from vegetation (during growing season); and (*e*) evaporation from permanent water bodies, if any (continuous). Moisture that supplies transpiration from vegetation is derived from water temporarily absorbed by the soil, which remains within the root zone during the growing season and is in excess of the wilting coefficient of the soil.

Stream flow is derived from immediate surface run-off (largely flood flow), and from water permanently absorbed by the soil which penetrates below the zone of plant roots and joins ground-water storage at the water-table, reaching stream channels later as ground-water discharge (normal or low-water flow). Stream flow as measured at any point in the channel is a net quantity after deduction of channel losses, of which transpiration from riparian vegetation is usually the most important. Channel seepage losses may also be important where the channel is permeable and does not cut the water-table. Sub-surface leakage is a function of geological formation, and with an impermeable bed-rock is negligible.

This complete view of the problem shows that the various physical characteristics of water-sheds, such as meteorology, vegetation, topography, soils, and geology, all have an influence upon stream flow, and also that several physical processes are involved, among which are evaporation, transpiration, and absorption. Of these processes, evaporation and transpiration have and are receiving much attention, while little study has been given to absorption, either as to rate or total quantity.

Absorption may be defined as the processes by which water enters the earth. These processes include flow into sink holes, influent seepage, absorption by capillary films, hygroscopic absorption, and inflow of atmospheric water vapor. Absorption from precipitation has two aspects, its rate, which is directly related to immediate surface run-off and flood flow, and its total quantity, which is directly related to ground-water discharge which maintains normal and low-water flow.

Rate of Absorption.—What bearing does the rate of absorption have upon the relation of forests to immediate surface run-off? Just this, that the rate of absorption depends primarily upon the permeability of the soil, and in so far as forests increase soil permeability, they decrease immediate surface run-off, and to the extent that they decrease soil permeability they increase such run-off. Forests growing in highly permeable soils free from clay, such as dune sand, pumaceous volcanic deposits, coarse alluvium, sandy loams with less than 4% clay, and even pure silt, such as certain loess deposits, have little or no effect upon surface run-off. Forests growing upon impermeable soils and those which contain more than 4% of clay, however, have an important effect upon surface run-off by decreasing it as a result of increasing the permeability of the soil. They accomplish this through the agency of roots and forest litter by the following processes:

1.—Roots penetrate the ground where they open cracks in hard formations, such as rock, and permeate softer formations, such as clay beds, with root systems that expand the soil and ultimately give channels for the entrance of water.

2.—Forest litter promotes the growth of soil fauna, such as worms, bacteria, etc., which are very effective in opening up an impermeable soil.

3.—Forest litter provides a source for humus, which is of greatest importance in opening a clayey soil, not only mechanically by mingling with inorganic soil particles and separating them, but chemically by the production (in the process of decay) of solutions which, acting in conjunction with natural lime in the soil, facilitate flocculation or the assembling of clay particles into soil crumbs.

4.—Forest litter protects inorganic soil particles from disturbance by the impact of falling rain drops or by the movement of flowing sheet water, and thus prevents the clay from segregating out into suspension with a resultant sealing action upon a soil which, although originally more or less impervious, has been opened up by Processes 1 to 3. As an illustration, a suspension of colloidal clay will completely seal a column of clean medium beach sand having a weight one hundred times that of the clay.

5.—Forest litter protects the surface of the clayey soil from consolidation by preventing the puddling produced by hoofs of grazing animals, or the crusting and consolidation caused by shrinkage with exposure to the sun. The degree of consolidation of a clay soil has a very marked effect upon its permeability.

6.—Finally, forest litter prevents erosion with its destructive effects, not the least of which is the sealing of permeable channels and overflow areas in alluvial filled valleys which are normally the intake areas for the economically important ground-water resources of these valleys. It is to be noted, however, that brush or grass may be as effective as forest litter in preventing erosion.

Total Quantity Absorbed.—The quantity of absorption is dependent not only on soil permeability, but also on the amount and distribution of rainfall during the annual cycle, the storage capacity of the soil in the zone of aeration available for plant discharge, and the transpirational requirements of vegetation. Of the total quantity absorbed, a portion, which may be termed temporary absorption, reaches only the zone of aeration from which it is drawn back and escapes into the atmosphere by evaporation from moist soil or transpiration from vegetation. The remainder is permanent absorption which penetrates to the water-table and enters the zone of saturation. The latter is the ground-water storage which feeds stream flow.

Soil moisture research with partly saturated soils has shown conclusively that movement of water through capillary films not within the capillary fringe of a water-table is so slow that re-adjustments do not take place during the annual cycle. As a result, downward movement of water through the zone of aeration does not occur unless and until moisture content is at or above that at which free gravity drainage commences.

Tests made in brush-covered areas of sandy and gravelly soils on valley floors in Southern California to determine total deficiency in moisture content within the root zone (8 to 18 ft) at the end of the dry season, indicate values of from 8 to 14 in. The same tests show that, including current losses during the rainy season by soil evaporation and transpiration, approximately 19 in. of seasonal rainfall is necessary before any considerable quantity of water will penetrate beyond the root zone. Data obtained by the U. S. Geological Survey in Central California have been analyzed by the writer, which indicate that for uncultivated fine sandy silt loam soils with natural grass and weeds, a depth of approximately 12 in. is required. No data are known to be available for mountain water-sheds with either brush or forest. The steeper slopes tend to cause surface run-off prior to full satisfaction of field capacity, while the shallower soils tend to reduce the available storage capacity in the soil. It is believed that for most mountain water-sheds the limit is between 12 and 20 in.

It is thus apparent that in a semi-arid region a considerable quantity of rain must fall before permanent absorption can commence. In fact, in regions where the average rainfall is between 12 and 20 in., it is probable that accretions to ground-water storage through deep soils supporting natural vegetation occur only in wet years. It should be recognized, however, that in irrigated areas permanent absorption from rainfall occurs when considerably less than these quantities have fallen; also, that in shallow soils which are less in depth than normal root penetration and are underlaid by bed-rock, or other impervious formation, gravity water may begin to accumulate at the bottom of the soil column and a water-table may form when lesser aggregate quantities of rain had fallen. Under the latter conditions, which are not unusual for mountain water-sheds, ground-water contributions might be made to adjacent stream channels from the superficial soil cover, as distinguished from that yielded by cracks, joints, and fissures in the bed-rock. The quantity thus contributed is controlled by the length, slope, and permeability of the intervening soil path. In view of the very slow movement of sub-surface water, the amount of such contribution is limited to that yielded by the lower slopes, that from upper slopes being intercepted and transpired during the growing season by intervening vegetation.

Turning again to forests, the following applications can be made of these facts regarding the amount of absorption:

(*A*) Under arid conditions, vegetation has no effect upon ground-water contribution to the summer flow of streams, except that roots of riparian, or stream-border, vegetation discharge ground-water that enters or leaves surface channels as influent or effluent seepage, and thus decrease the flow.

(*B*) Under humid conditions, forests and other vegetation growing upon permeable soils decrease ground-water contribution to normal and low-water flow by transpiration, the quantity of the latter depending upon the requirements of the individual species. Deciduous forests consume more than grasses, but evergreens possibly not as much. Forests and brush growing upon soils containing appreciably more than 4% clay, however, offset this decrease by increasing the permeability of the soil, and in clayey soils may actually increase normal and low-water flow.

(*C*) Under semi-arid conditions (wet winter and dry summer), forests and other vegetation growing on deep soil decrease ground-water contributions to summer flow by transpiration as compared with bare soil. As precipitation and melting snow in such regions occur principally during the dormant period when transpiration is very slight, the decrease is less than the transpiration requirements of the species. The measure of the decrease is approximated by the quantity of soil moisture within the root zone and between field capacity and wilting coefficient. This follows from the fact that transpiration supplied by rain which falls during the growing season is largely an offset against evaporation from bare soil during the same period. The moisture depletion varies from 8 to 14 in. for brush on deep sandy soil in valleys. These limits would be less for shallow soils and for deep clayey soils, and are probably not out of line for forests in mountain water-sheds. For a season when rainfall plus current evaporation from bare soil exceeds these amounts during the period of replenishment of soil moisture, the substitution of forests for bare land would have no effect upon summer flow derived from ground-water.

Summarizing this discussion, and with particular reference to absorption:

1.—Forests decrease immediate surface run-off from precipitation: (*a*) By interception and evaporation from foliage, especially in summer; (*b*) by retarding the rate of melting snow lying on impermeable soil or rock; and (*c*) by increasing the permeability and, hence, the rate of absorption of soil containing more than approximately 4% of clay.

2.—Forests have no effect upon immediate surface run-off from precipitation or melting snow when growing upon highly permeable soil, nor under conditions of such long-continued or intense rainfall that the forest litter becomes saturated.

3.—Forests decrease summer stream flow derived from ground-water storage: (*a*) In humid regions by the full amount of seasonal transpiration on highly pervious soil areas and by lesser amounts on soils containing more than 4% clay to the extent that permeability is increased; and (*b*) in semi-arid regions by a portion of the transpiration not exceeding from 12 to 20 in., with smaller limits under various conditions of soil depth and permeability. This assumes that evaporation by interception offsets evaporation from bare soil without forest.

4.—Forests on clay soil may increase summer stream flow derived from ground-water storage by increasing soil permeability.

5.—Forests decrease total annual run-off as compared with that from bare soil, but as compared with brush or grass they decrease the run-off only when the total of forest transpiration and interception exceeds that of the brush or grass.

While these conclusions do not differ from those of the authors for the two small water-sheds studied, they indicate the errors that might result from too general an application.

W. G. Hoyt,[65] M. Am. Soc. C. E., and H. C. Troxell,[66] Assoc. M. Am. Soc. C. E. (by letter).—The earnest consideration given to this paper, as indicated by the numerous contributions submitted, is appreciated. It is also extremely gratifying that, with one or two exceptions, the discussions have either agreed with the conclusions and deductions of the writers or have contained constructive criticism.

Unfortunately, Mr. Bates looks upon the paper as an attack upon the forests and the forester instead of a contribution to the science of hydrology. He discusses soil, geology, meteorology, and "Nature's function" in an attempt to prove that the facts found in specific areas are not generally applicable. Reliable statistical information, however, collected under rigid scientific control in areas presenting widely different conditions, must be held to outweigh opinions such as he presents, no matter how firmly held or enthusiastically advocated.

No question has been raised by any member of the Society as to the method of analysis used, and no one has disputed the conclusions that on the two areas under consideration: (1) Forests and brush cover substantially lower stream flow at practically all seasons and all stages, including flood, minimum, and total flow; (2) forests and brush cover substantially lessen erosion where readily erodible material is available; and, (3) new vegetative growth exercises an effect similar to the original cover in retarding erosion, but without so detrimental an effect on annual run-off and summer low-water flow.

Messrs. Lowdermilk and Gilman deplore, as do the writers, the absence of precipitation records in the Southern California area. Hydraulic engineers will realize readily, however, that the method of analysis used is independent of rainfall records, but takes into account all normal differences in meteorological conditions that are reflected in run-off from the two areas.

Additional information concerning the climatic conditions and the geology would be most helpful in a search for the relative influence of causes contributing to the results. The lack of such information does not impose a serious limitation on the validity of the conclusions, nor does it cast doubt on the aforementioned facts.

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However, the fact that Mr. Gilman apparently believes that the precipitation record for Mount Wilson had been used in computing the normal run-off of Fish Creek indicates that the method of determining the normal run-off, described in the paper, may not be fully understood by all readers.

Students of hydrology in Southern California have long known that there is not a simple direct relationship between run-off and rainfall. For example, a 2-in. rainfall, if concentrated within an hour, may cause flood run-off, but the same rainfall distributed over 24 hours would cause little if any run-off. For this reason, unless the intensity of the rainfall and its relation to run-off are known, the rainfall record is of little value in determining the normal run-off of Fish Creek.

The daily relationship of the discharge for Santa Anita and Fish Creeks was plotted for each day for the period prior to the fire (1917–1924). These points indicated a fairly well-defined, simple, direct relationship of the run-off from the two areas, as exemplified in the following tabulation, which records various simultaneous flows (in second-feet per square mile) that occurred during the winter months of January to March, 1917 to 1924:

Daily discharge, Santa Anita Creek	Daily discharge, Fish Creek	Daily discharge, Santa Anita Creek	Daily discharge, Fish Creek
40	82	2	2.6
20	37	1.0	1.2
10	16.5	0.5	0.58
5	7.4	0.2	0.21

The relationship was so well-defined for the period prior to the fire that there is good reason to believe that it would have been maintained for the period following the fire if the area had not been burned over. This relationship, wholly independent of records of precipitation, was used to compute the normal run-off from Fish Creek after the fire.

The rainfall records from Mount Wilson and Santa Anita Ranger Stations were included merely to show how little of the rainfall appeared as run-off at the gauging stations. For example, in Southern California, a seasonal rainfall of 20 in. produces, as a rule, a run-off of less than 2 in., and a seasonal rainfall of 30 in., generally less than 10 in.

Mr. Lowdermilk questions whether there is any relationship in rainfall for the drainage areas of Fish Creek and Santa Anita Creek and presents data showing the variation in seasonal rainfall at points ranging from less than 1 000 ft to more than 5 000 ft in altitude, and from less than 1 mile to more than 15 miles, from these drainage areas.

In Southern California many storms vary with the altitude; some affect only the higher altitudes, some produce rain only in the lower reaches, and others are general, giving much the same rainfall at all altitudes. Furthermore, storm rainfall varies from west to east and from east to west, depending on the direction from which the storm approaches. Thus, it would scarcely be expected that even an approach to uniformity in seasonal rainfall could be attained.

On Fig. 17 are plotted the relationships between storm rainfall for the period, 1917–1930, for Mount Wilson, for the San Gabriel power-house, near the mouth of Fish Creek, and for the Sierra Madre, just west of the mouth of Santa Anita Creek. For the lesser storms, Mount Wilson has sometimes more rainfall than the other stations and sometimes less, but as the storm rainfall increases, the relationship is better defined. In almost every storm the rainfall at the Sierra Madre, although generally greater than at the power-house, bore a relationship to that at Mount Wilson substantially parallel to that borne by the rainfall at the San Gabriel power-house. These comparisons should resolve Mr. Lowdermilk's doubt as to the general similarity of rainfall graphs for the two areas; but, after all, the most convincing evidence of rainfall relationship between these areas for which a direct record is meager, is found in the well-defined run-off relationship, for which the direct record is ample, continuous, and areally integrated.

The additional detailed information by Mr. Lowdermilk showing the composition of chaparral cover in Southern California is a welcome contribution to the discussion. It is regretted that the writers did not have access to unpublished Forest Service records, and that, therefore, their description of vegetation was incomplete, but after all the names of vegetative species and knowledge of their characteristics, like records of rainfall, are useful aids to interpretation of results but cannot change records of run-off.

Readers should realize that the Wagonwheel Gap experiments were extensive and exhaustive and that they cost tens of thousands of dollars. The Southern California study cost almost nothing, being incidental to routine stream gauging. The latter study lacked many of the details of rainfall, temperature, soil cover, and sediment observations of the former and, therefore, failed to afford as extensive a field for study of the relative effects of underlying causes. The effects of change in cover on run-off, however, were recorded as effectually in the Southern California area as in Colorado, and the major conclusions may be drawn with as much assurance on one area as on the other.

Facts or conclusions as to small specific areas have little value unless their application to other areas can be fairly well demonstrated. Mr. Bates presents the view that the Wagonwheel Gap and Southern California areas are so different from the greater proportion of forested areas in the United States that no conclusion generally applicable can be drawn from them.

In so far as the Wagonwheel Gap area is concerned the writers must dissent from this view. In 1922, Mr. Bates stated[67]:

> "It was, therefore, with a full realization of the need for experimental proof and in the hope of satisfying the most critical that in 1909 the Forest Service undertook to initiate a project which should produce results bearing particularly on the conditions typical of vast areas then included in national forests of the West."

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[67] "Stream Flow Experiment at Wagonwheel Gap, Colorado; Preliminary Report on Termination of First Stage of Experiments," *Monthly Weather Review, Supplement 17*, p. 2.

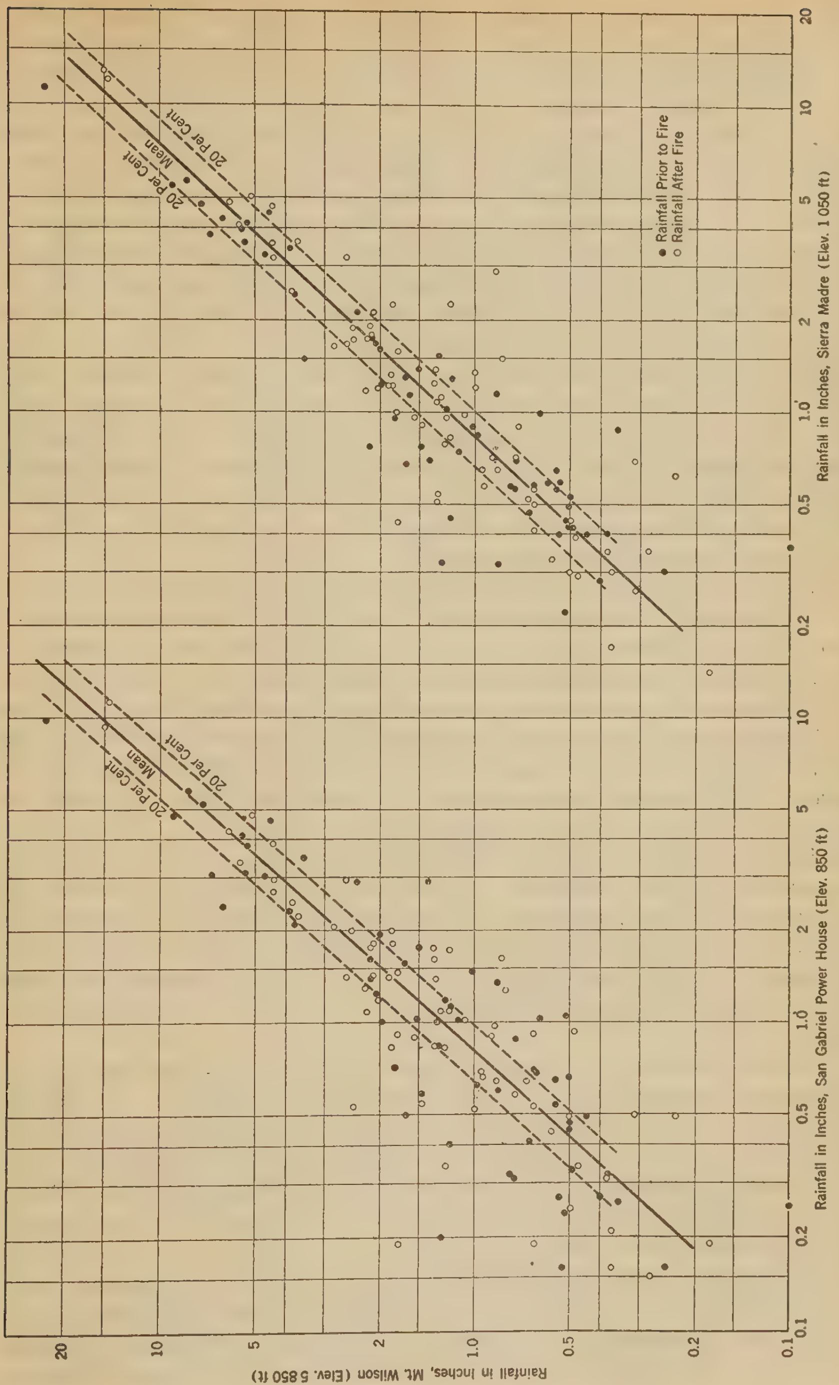

FIG. 17.—RELATION BETWEEN STORM RAINFALL; MOUNT WILSON, SAN GABRIEL POWER HOUSE, AND THE SIERRA MADRE.

In seeking areas where studies should be made, the following factors were considered by the Forest Service: Areas should be (1) contiguous; (2) small; (3) representative of a fair average for the National forests of the Rocky Mountain region; and (4) accessible. Credit for the discovery of such an area in Wagonwheel Gap is given to a Forest Assistant, and after the area had been visited by several Forest officials, including the Assistant Forester, its selection was approved. After close acquaintance with the area for more than twelve years Mr. Bates considered that "excellent judgment" had been used in its selection for the purpose of satisfying the most critical of the correctness of a preconceived conclusion. It scarcely seems fair to the public, when that conclusion is disproved in part, to suggest now that the area is extreme and not fairly representative.

Many of the other discussers take what seems to be a more rational attitude—that identical results in Colorado and California under such widely different circumstances constitute evidence of the probable universality of the conclusions, in the absence of trustworthy experiments to the contrary. All have stressed the desirability of additional exhaustive studies, and in that opinion the writers concur fully. Experiments in a humid area and on a fairly impervious soil are needed especially.

By processes of deduction, Messrs. Lowdermilk, Lee, and others, conclude that the increase in the summer flow from the denuded Colorado area resulted from decreased interception by the forest cover, and that the corresponding increase in the Southern California area resulted from decreased transpiration. No attempt was made in the paper to explain in detail the reasons for the increase, and, in the light of present knowledge, the writers do not believe that explanation is altogether possible or, if possible, that it greatly matters. All vegetation intercepts; all vegetation transpires. Removal of vegetation will diminish both interception and transpiration. If it is true that interception was in the ascendency in the Colorado area and that transpiration was in the ascendency in the Southern California area, that is just another of those differences in regional conditions that suggest the universality of conclusions that were identical for the two fields of experiment. The essential and undisputed fact still remains that with a diminution of the vegetative cover the annual flow and the low-water flow increased on both areas.

Additional studies have been made, however, and from these the writers believe that the deductions of Messrs. Lowdermilk and Lee are not entirely correct. To the best of their ability the writers have separated the run-off caused by summer storms from the ordinary non-storm run-off for three years prior to deforestation and for three years afterward. Some of the results of these studies are shown graphically in Figs. 18 to 21, inclusive; others are shown in Table 22.

The data in Table 22 fail to disclose that increase in summer storm run-off resulting from rainfall which should follow decrease in interception by reason of deforestation. The hydrographs in Figs. 18 to 21 are especially instructive in showing the increased non-storm summer flow after deforestation, a result that should logically follow decreased transpiration. It is clearly evident that there was more ground-water available to sustain summer run-off in the Wagonwheel Gap area after deforestation than before, and it is more than likely that both decreased interception and decreased transpiration contributed to this result, although the latter seems to have been the major cause.

TABLE 22.—STORM RUN-OFF AND RAINFALL, IN INCHES, JULY AND AUGUST, AT WAGONWHEEL GAP, COLORADO

Season	AREA *A*			AREA *B*			Ratio, storm run-off, Area *B* to Area *A*
	Rainfall	Storm run-off	Ratio	Rainfall	Storm run-off	Ratio	
(*a*) BEFORE DEFORESTATION OF AREA *B*							
1912	5.86	0.06441	0.010	5.81	0.05088	0.0088	0.790
1913	4.67	0.04415	0.0095	4.94	0.03902	0.0079	0.884
1914	7.10	0.09504	0.0134	7.47	0.05877	0.0079	0.618
(*b*) AFTER DEFORESTATION OF AREA *B*							
1920	3.32	0.03109	0.0094	2.74	0.01927	0.0070	0.620
1921	6.42	0.04653	0.0072	6.25	0.02758	0.0044	0.593
1922	5.40	0.07170	0.0133	4.88	0.06044	0.0124	0.844

Mr. Lowdermilk may also have gone too far in his assumption that all the gain in summer flow after the burning of the Southern California area resulted from decreased transpiration by canyon-bottom vegetation. Mr. Blaney's work in Coldwater Canyon showed that canyon-bottom vegetation during the 6-month period used 45 acre-in. per acre, or 238 acre-in. per mile of canyon. On the assumption that transpiration by riparian vegetation above the falls in Fish Creek Canyon proceeded before the fire at the rate per linear mile indicated for Coldwater Canyon, that such transpiration was wholly eliminated after the fire, and that there were no compensating water losses by reason of increased evaporation, it might be freely admitted that change in riparian vegetation could account for the entire recorded change in summer run-off. The first of these assumptions is speculative, however, and the others are untenable. Riparian vegetation would recover rather quickly from the effects of the fire, and if its destruction had caused increased summer flow this effect would soon be lost.

The increase in summer flow, however, was well sustained for several years, suggesting that no such close relationship existed between cause and effect here as was assumed by Mr. Lowdermilk. Moreover, there would be undoubted although quantitatively unknown increased evaporation to compensate in part for the effects of decreased transpiration by riparian vegetation. The continued maintenance of summer flow strongly suggests that these factors—transpiration and evaporation—were in fact compensating to a considerable degree. In the light of all the available facts, it seems that decreased transpiration was only one of several factors contributing to increased summer flow after the fire.

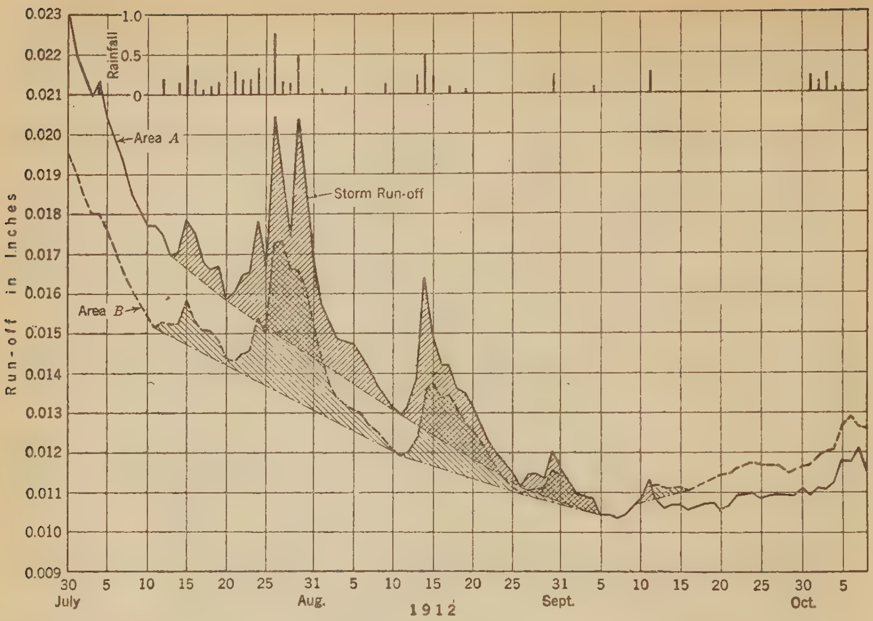

FIG. 18.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1912 (BEFORE DEFORESTATION).

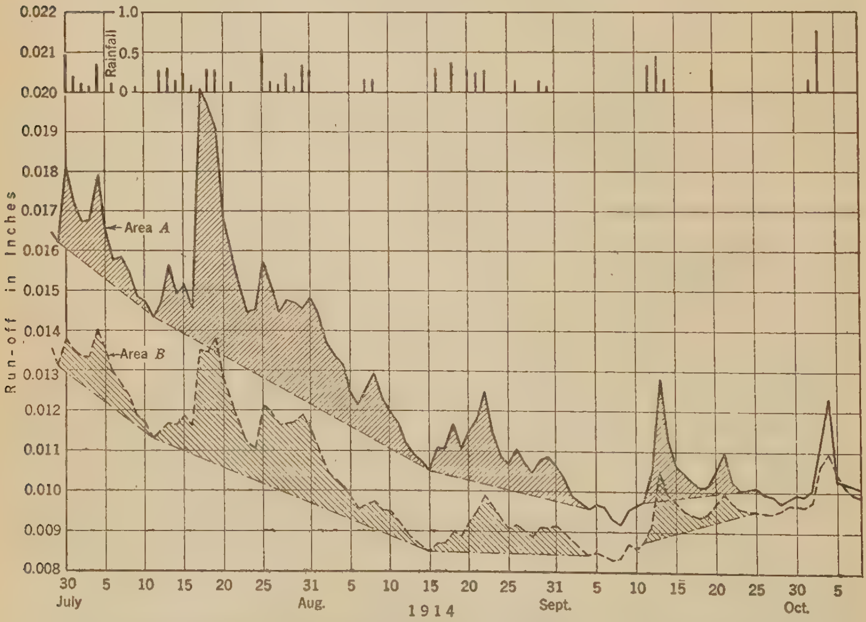

FIG. 19.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1914 (BEFORE DEFORESTATION).

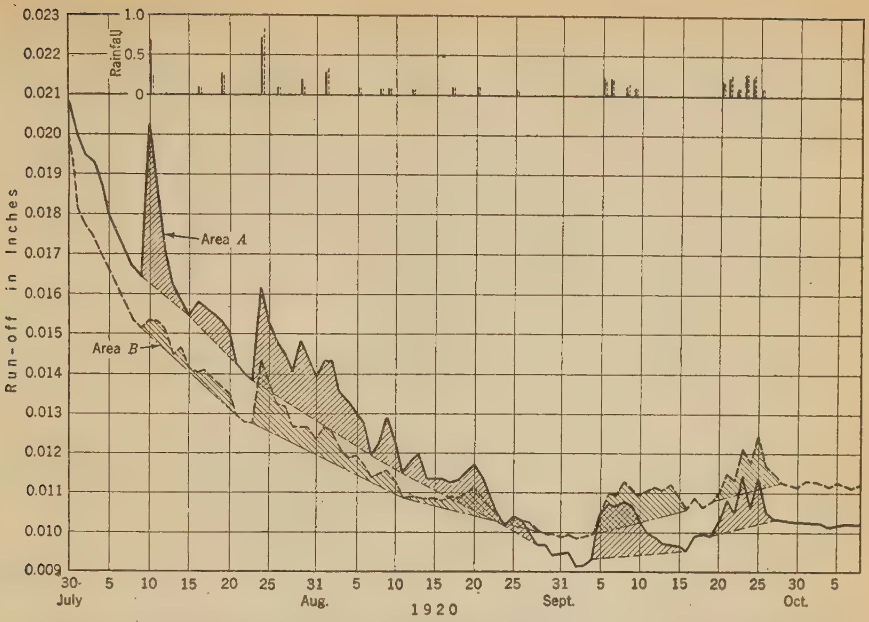

FIG. 20.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1920 (AFTER DEFORESTATION OF AREA *B*).

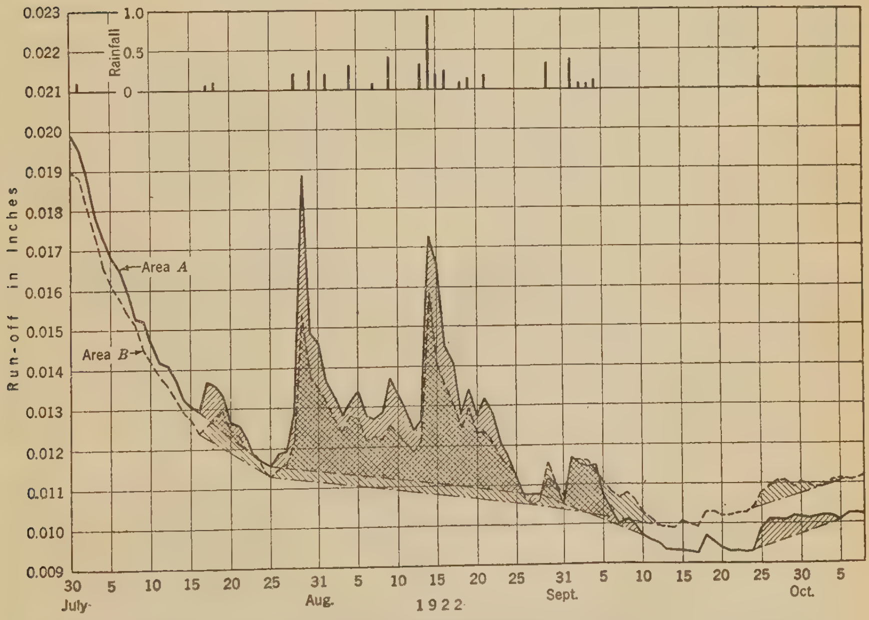

FIG. 21.—STORM RUN-OFF, WAGONWHEEL GAP, COLORADO, JULY TO OCTOBER, 1922 (AFTER DEFORESTATION OF AREA *B*).

Because of the many uncertainties involved the writers have refrained from attempting to distribute the source of the summer gain between the burned-over canyon-bottom plant life and the chaparral-covered canyon walls in Southern California, or between the interception losses, transpiration losses, and increased ground storage in the Wagonwheel Gap area.

Mr. Lee presents an excellent discussion of the part played by forest litter on run-off and absorption. Forest litter also forms the basis for the increased absorption in Mr. Sonderegger's "super-forestation." The recent experiments by Mr. Lowdermilk[68] have contributed to knowledge of the effect of litter on stream flow. In his discussion, Mr. Lowdermilk cites these experiments as a basis for the following statement:

> "The forest cover, especially the litter which it produces, furnishes conditions which make for maximum possible water and erosion control on watershed surfaces."

In so far as erosion is concerned the writers agree with this statement. They question, however, the use of the word, "water," in this broad conclusion if by "control" Mr. Lowdermilk implies that the presence of litter is reflected in an increased water yield, or increased low-water flow.

His studies show clearly that litter decreased direct run-off and, therefore, must have increased absorption. As Mr. Lee points out, however, part of the water passing into the ground is only "temporarily absorbed," being held in the zone of aeration, from which it escapes into the atmosphere by evaporation or transpiration from vegetation. The remainder is "permanently absorbed" penetrating to the water-table and entering the zone of saturation. This part is the ground-water storage that feeds stream flow. Mr. Lowdermilk fails to record the quantity of absorption that passed through the soil column in his tanks, and even if it had been so recorded it would not be indicative of Mr. Lee's "permanent absorption" under natural conditions, because the tanks were entirely artificial, in that they contained no vegetative cover to transpire what under natural conditions would have been "temporarily absorbed."

To have a litter-covered soil in Nature requires a canopy of brush or trees. As stated by many contributors, interception by and evaporation from such a canopy may range from 10 to 40% of the rainfall. Under natural conditions this interception might readily offset any increased absorption such as is inferred from results obtained in barren, artificially litter-covered tanks. Furthermore, the tanks used by Mr. Lowdermilk were set at a 30% slope, whereas in the Southern California area, as shown in Table 23, the slopes are much greater.

It will be seen from Table 23 that the major parts of both the Santa Anita Creek and Fish Creek areas have slopes exceeding 60 per cent. The slope at which the litter has the maximum effect on run-off was not determined by Mr. Lowdermilk, but it is possible that results from tanks with 30% slopes have little bearing on natural slopes exceeding 60 per cent.

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[68] "Influence of Forest Litter on Run-Off, Percolation, and Erosion," by W. C. Lowdermilk, *Journal of Forestry,* Vol. 27, No. 4, April, 1930.

After all, the test of the effect of litter on low-water stream flow is not how much of the rainfall percolates into the root zone, but rather how much of the rainfall passes into the zone of saturation, which sustains stream flow during non-storm periods.

TABLE 23.—SLOPE OF SOIL AND CANYON WALLS OF SANTA ANITA AND FISH CREEK DRAINAGE AREAS

Slope	SANTA ANITA CREEK		FISH CREEK	
	Square miles	Percentage of drainage area	Square miles	Percentage of drainage area
Greater than 80%	1.5	14.3	0.5	7.7
From 80 to 60%	5.7	64.3	5.6	86.2
Less than 60%	3.3	31.4	0.4	6.1
Total	10.5	100	6.5	100

The field tests made by Messrs. H. F. Blaney and C. A. Taylor[69] on rainfall penetration on the valley floor present the results from actual rainfall in 1927–28, based on analyses of soil moisture. There was no direct run-off from any of the plots during the season involved. Table 24 shows the distribution of the rainfall.

TABLE 24.—EVAPORATION, TRANSPIRATION, AND RAINFALL PENETRATION IN SOUTHERN CALIFORNIA

Location		Soil	Crop	Dates	Rainfall, in inches	Evaporation and transpiration, in inches	Penetration below root zone, in inches	Stored in soil, in inches
Anaheim..	Plot A-1.	Hanford fine sandy loam	Clean	October 1, 1927 to April 15, 1928	12.39	6.0	1.5	4.9
	Plot A-2.	Hanford fine sandy loam	Cultivated grass	October 1, 1927 to April 15, 1928	12.39	12.0	0	0.4
Ontario...	Plot D-1.	Hanford sand underlaid by silt loam	Clean	October 1, 1927 to April 17, 1928	11.52	4.7	0	6.8
	Plot D-2.	Hanford sand underlaid by silt loam	Cultivated grass	October 1, 1927 to April 17, 1928	11.52	7.9	0	3.6
Ontario...	Plot E-1.	Hanford sand	Clean	October 1, 1927 to April 17, 1928	12.79	6.8	0	6.0
	Plot E-2.	Hanford sand	Cultivated grass	October 1, 1927 to April 17, 1928	12.79	9.6	0	3.2
Glen Avon Heights	Plot G-1.	Placentia loam	Clean	October 1, 1927 to April 13, 1928	12.18	8.1	0	4.1
	Plot G-2.	Placentia loam	Cultivated vetch	October 1, 1927 to April 13, 1928	12.18	10.5	0	1.7

In each of the tests noted in Table 24, more rainfall was stored in the clean cultivated plots than in those bearing vegetation. The depth of the root zone was from 6 to 11 ft, and the initial deficiency of soil moisture in the root zone ranged from 4.6 in. on Plot A-2 to 13.2 in. on Plots D-1 and D-2. As soon as the initial deficiency of soil moisture is overcome, rainfall penetrates below the root zone and becomes a contribution to ground-water.

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[69] "Rainfall Penetration and Consumptive Use," by Harry F. Blaney, Assoc. M. Am. Soc. C. E., and C. A. Taylor, *Water Resources Bulletin 33*, California Dept. of Public Works, 1930.

The soil conditions, as so ably outlined by Messrs. Mead and Lee, would have a major effect on storm run-off and absorption. On soils of the more pervious material, forests have little effect on direct run-off. This is amply illustrated by Figs. 18 to 21, which show that the removal of the forest cover in the Wagonwheel Gap experiment did not perceptibly increase the storm run-off following summer rains.

"Super-forestation," as Mr. Sonderegger states, would "keep the water in the water-shed," but retarding storm run-off by means of plant life does not imply conservation of water. As Mr. Rule states, it results in "a reduction in the residual quantity available for capture by Man."

Under the climatic conditions existing in the Santa Ana River Valley and the Coastal Plain in California, according to Messrs. Blaney and Taylor:[70]

> "The results of the last three years' work on brush plots indicate that a seasonal precipitation of at least 19 inches is necessary before any material amount of water will penetrate below the brush root zone of the valley floors. A seasonal rainfall of less than 19 inches is usually consumed by the brush-covered area before the ground water receives any increment."

Messrs. Blaney and Taylor also show that for equal rainfall more of the rainfall passes below the root zone in bare soil than in soil covered with plant life and that deficiency of soil moisture is much less in bare soils than in brush-covered areas.

Recent excavation for roads in chaparral areas in Southern California has uncovered root systems that have extended as deep as 30 ft into shattered rock in search of water. Only during years of exceptional rainfall, such as 1921–22, could important accretions from rain be made to ground-water supplies in this region.

The writers are in harmony with Mr. Rule's valuable contribution showing the relation existing between vegetal cover and storage soil moisture. The facts brought out in several of the discussions tend to confirm Mr. Rule's suggestion that in so far as the Southern California area is concerned a large part, but not necessarily the major part, of the increased summer flow may be credited to reduction in the consumptive use of water by canyon-bottom vegetation. The writers stated, that "some of the alders and sycamores adjacent to the stream escaped burning." Mr. Lowdermilk declares that this statement is entirely misleading, as this condition prevailed only below the falls in Fish Canyon, while above the falls all canyon-bottom vegetation was killed. Apparently, less canyon-bottom vegetation escaped burning than was indicated to Mr. Lowdermilk by the word, "some."

The studies by Messrs. Blaney, Taylor, Nickle, and Young, as described by Mr. Blaney, are of outstanding value in showing the extent of water losses through transpiration by hydrophytic, or more properly phreatophytic, canyon-bottom vegetation. These studies show gains of more than 100% in summer flow through the elimination of transpiration losses in canyon bottoms having widths of only 30 to 50 ft. It seems apparent that considerable gains could be obtained through the reduction of similar losses now occurring in streams, such as the Santa Ana and others which meander for miles through wide brush and tree-covered areas. Pumping ground-water into conduits constructed across such areas, thus lowering the water-table below the root zone and reducing losses by evaporation and transpiration, may be economically sound in areas where water-rights are worth between $100 000 and $200 000 per sec-ft.

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[70] "Rainfall Penetration and Consumptive Use of Water in Santa Ana River Valley and Coastal Plain," by H. F. Blaney, Assoc. M. Am. Soc. C. E., and C. A. Taylor, *Bulletin 33*, California Dept. of Public Works, 1930, p. 21.

Mr. Barrows touches lightly on one phase of the question which may be particularly applicable to New England and the North Atlantic region. Statements of foresters indicate that many water-sheds in this region have been heavily reforested with the sole idea of increasing water supply for municipal use. As Mr. Barrows states, and as the experiments clearly indicate, "it may be worth while for other reasons, * * *, but certainly as regards increasing the water supply the results are likely to be detrimental rather than helpful."

Mr. Baker believes that the "superficial run-off + ground storage" would be a "minus" after deforestation, and, consequently, that "precipitation" in his basic formula would be reduced. Here, again, the assumption is made that increased direct run-off necessarily connotates reduction of ground-water storage. It is an important fact that on the Wagonwheel Gap area the normal flood run-off was held temporarily as ground-water. The flood run-off increased after deforestation; consequently, more water must have entered the soil column after deforestation. The work of Messrs. Blaney, Taylor, and their associates in Southern California, already referred to, seems to substantiate this point.

Mr. Baker apparently accepts the conclusion as sound for areas having a "marine type" of precipitation, but raises a question as to the possible detrimental effect of deforestation of several thousand square miles having a "continental type" of precipitation.

The effect of plant life on precipitation is still a debatable question, to which there seems to be no definite answer. It appears unlikely, however, that the loss, if any, in the "continental type" of rainfall due to a change in vegetative cover from trees to smaller species would offset the gain in water supply. It would seem as logical to suppose that the present practice of irrigation on 30 000 sq miles of arid land in the West by changing the cover from the desert type to farm crops would have an effect upon precipitation as to suppose that a change in cover on several thousand square miles in the Southeast would have a serious effect upon agriculture. Incidentally, the Wagonwheel Gap area is probably as far removed from a "marine type" of climate as any other area in the United States that could be selected.

Comment seems unnecessary with respect to claims that deforestation has changed climate or that forestation increases precipitation. These and similar claims are believed to be incapable of either proof or disproof. Some assert that removal of the cedars of Lebanon brought destructive floods and resultant erosion[71], but according to the Bible long before those cedars were removed the greatest flood of the world took place and was the direct result of abnormal precipitation, as are the floods of to-day.

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[71] Partnership of Wood and Water," by Hans Burger, *American Forests*, April, 1932, p. 147.

The studies by Messrs. Burr, Chittenden, Harts, Hazen, Mead, Stevens, and others already cited indicate, in a general way at least, that over broad areas no decreases in stream flow can be attributed to deforestation. To their conclusions may be added those recently reached in Ohio by C. E. Sherman, M. Am. Soc. C. E., who after a study[72] of run-off measurements made by Mr. D. S. Bates for the Ohio Canal Commission in 1825, when "not one-half of the 40 000 square miles of our Ohio territory is yet settled; more than five-sixths of it remains uncleared and uncultivated," concludes, in part, as follows:

> "The Ohio Canal Commission reports which give early stream records make surprising reading for those interested in the effect of forests on drought, for it is difficult to read them carefully without concluding that at many places in Ohio reforestation would have little effect in maintaining good stages of water during severe drought."

Mr. Lowdermilk states that the late Professor Swain, "deserves to rank in authority much higher than Colonel Chittenden." The writings and statements of both these distinguished and able engineers were made prior to the Wagonwheel Gap experiments. Professor Swain fully realized the need of tangible evidence when he discussed the late Colonel Chittenden's masterpiece in 1909. He states in his discussion[73] that,

> "In order to demonstrate quantitatively beyond peradventure the effect of forests, it would be necessary to have the same area under identical conditions in every respect, except that at one time it should be wooded and at another time deforested."

It is possible only to surmise to what extent the opinions of both these men would have been changed, if at all, had the results of the Wagonwheel Gap and Southern California experiments been available to them. Would they have ignored the records, or would they have changed their opinions to agree with the records? To the extent that the conclusions and deductions of both agree with facts as now known, they should be accepted; to the extent that they are shown to be fallacious, they should be ignored.

Mr. Smead mentions the investigations of 1911 and 1912 in the White Mountains which were carried on by the U. S. Geological Survey under the provisions of the Weeks Act. In these investigations ten nearly contiguous areas in New Hampshire were selected. No final report of the investigations has been prepared by the Geological Survey, but in a preliminary statement[74], the following conclusion was reached, based on comparisons made on two successive floods occurring on two of the ten streams during one spring period:

> "The results of the Burnt Brook-Shoal Pond Brook studies are held to show that throughout the White Mountains the removal of the forest growth must be expected to decrease the natural steadiness of dependent streams, during the spring months at least."

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[72] "Ohio Stream Flow," Ohio State Univ. Studies, Vol. 1, No. 5, 1932, p. 164.

[73] *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 365.

[74] Mimeograph report of thirty-one pages dated June 4, 1912, signed by George Otis Smith, Director.

On this conclusion rested largely the expenditure of several millions of dollars in the acquisition of National Forests in the East under the Weeks Act. Since the preliminary statement was issued considerable time and effort have been expended in a study of the original run-off records collected on the ten areas to determine whether it was possible to separate the effects of the inherent basin characteristics—soil, geology, shape, slope, and aspect—on the run-off from the effect of the vegetative cover.

To date this separation has not been possible, although apparently the inherent basin characteristics have more effect upon stream flow than the vegetative cover. In other words, observed differences may be the result of any one condition or combination of conditions, into which forest cover may not enter. This fact, which is also well supported by exhaustive studies referred to by Professor Mead and others and which is commonly accepted by all students of hydrology, renders studies without "control" substantially valueless. It is for this reason that the writers stress the value and importance of the type of studies carried on in Colorado and Southern California. In these areas changes from the normal are shown to be directly related to changes in the vegetative cover.

In all prior experiments, such as those mentioned by Mr. Smead, many described by Mr. Zon,[75] or the much quoted Swiss experiments,[76] it is impossible properly to ascribe reasons for noted differences in stream flow, because of a lack of "control." There is no doubt that further studies through the co-operative effort of the ecologist, forester, soil technologist, hydraulic engineer, and geologist, are necessary to determine the best type of vegetative cover for different types of soil, topography, and rainfall.

Such studies may well follow the method used at Wagonwheel Gap. In fact, in that area there is an excellent opportunity for an interested agency to study the effect of the new growth of aspens on stream flow as compared with the original cover, which consisted largely of conifers. The Utah experiment by Forsling referred to by Mr. Stabler was made along the lines adopted for the Wagonwheel Gap area. It was not wholly satisfactory, however, on account of the inability to hold an area in a semi-arid country uniform as to minor surface irregularities and vegetative cover. Still, as Mr. Stabler points out, the Forsling experiment tends to corroborate conclusions in the main paper and to extend their application. The type of experiment outlined by Mr. Blaney is satisfactory so long as it is conducted in rock-walled canyons having no ground-water inflow between the measuring sections. In areas where ground-water inflow takes place complications are introduced, which make it extremely difficult to determine losses resulting from vegetation. Controlled experiments on small plots, to which reference is made by Messrs. Bates and Lowdermilk, are unquestionably of value, but there is grave doubt as to their application to natural water-sheds in so far as water yield or net erosion is concerned.

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[75] "Forests and Water in the Light of Scientific Investigation," by Raphael Zon, Forest Service, U. S. Dept. of Agriculture, 1927.

[76] "Untersuchungen über den Einfluss des Waldes auf den Stand der Gewässer," by Arnold Engler, Schweizerischen Zentralstadt für das forstliche Versuchswesen Mitteilungen, Vol. 12, 1919, Zürich, Switzerland; also "Partnership of Wood and Water," by Hans Burger, *American Forests*, April, 1932, p. 147.

With particular reference to conditions in Southern California, Mr. Cecil states,

"This vast sum [well over $1 000 000] is expended [annually] not for the purpose of protecting a commercial timber supply or, for the most part, even a growth of tree species. The cover is, in the main, chaparral * * * [maintained] on the assumption that: (1) It is a conserver of water; (2) it controls erosion; (3) it retards flood run-off; and (4) recreational value would be impaired by its removal."

That the chaparral cover "conserves water" has not been demonstrated. The question at issue, therefore, is whether chaparral or some other cover best fulfills the other intended purposes.

The balance between soil, topography, run-off, rainfall, and vegetation in natural undisturbed water-shed units does not necessarily represent optimum conditions of water supply. It is fairly well demonstrated that a minimum of vegetative cover will be reflected in a maximum water yield. Mr. Zon[76] states this same principle: "The more highly developed the vegetal cover the faster the moisture is extracted from the soil and given off into the air. In this respect, forest is the greatest desiccator of the soil." If chaparral represents a minimum vegetal cover that will diminish or retard erosion and increase the absorptive capacity of the soil, and if the cost of protecting it from fire is commensurate with the benefits received, then the community expenditure of more than $1 000 000 per year for chaparral protection is justified.

Mr. Tracy I. Storey,[77] presents evidence indicating that when burning of certain chaparral areas was practiced in California, herbaceous vegetation was plentiful, "but since burning has been prohibited herbaceous vegetation and grasses are screened out by the growth of chaparral." Apparently, some thought should be given to the advisability of continuing a policy of retaining, through rigid control at vast annual expense, an inflammable cover that is useless for man or beast when it appears that, in parts of California, at least, restricted burning of chaparral areas will permit the growth of herbaceous grasses that would have economic value for grazing and also a water-shed "protective value." The same thought is also expressed by Mr. Willoughby.

On account of the necessity for economy in cost of publication, the original paper was greatly condensed. Necessarily, in so doing, as Mr. Cecil states, "several factors * * * that are important * * * have been overlooked entirely, and others have not been given proper value in arriving at a conclusion."

With the possible exception of erosion it is believed that in the main any omissions from the original paper have been covered in the various discussions. Although the original paper dealt principally with the effects of forests on stream flow, it was realized that the subject of erosion is intimately connected with the problem under consideration. The writers stated, that: "In the Southern California area complete denudation increased erosion as a direct result of the increased surface run-off" and "deposition of eroded material and ash carried by the streams during the first year after the fire was materially injurious to agricultural lands and transportation rights of way below the canyon." Unquestionably, the eroded material was a source of expense and inconvenience. Floods and attendant erosion, however, have been occurring in this region for centuries prior to the settlement of Los Angeles County.

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[77] "Factors Influencing Wild Life in California, Past and Present," *Ecology*, October, 1932, p. 323.

E. C. Eaton, M. Am. Soc. C. E., to whom Mr. Cecil refers, states,[78]

> "Take Los Angeles County as an example; it consists of a broad sloping valley area with a background of high mountains roughly equal to the valley area, from which ages of erosion have built deltas to known depths of more than 1 000 ft. * * * The tremendous area and great depth of the gravel cones are in reality of the greatest benefit to this southern County. * * * These underground reservoirs are responsible for the entire early development of the County and supply two-thirds of the water used in the County to-day."

Messrs. Cecil, Gilman, and Lowdermilk infer that the increased run-off made available through diminution of forest cover is non-usable and wholly destructive. Their inference may not be correct.

It is a well-known fact that a large part of the water supply in Southern California is pumped from underground basins and that as a means of increasing the ground-water storage spreading grounds have been established at various points, generally along the mountain fronts. Experience has shown that water which carries suspended matter is not readily absorbed on these spreading grounds. If the present spreading grounds were the sole entry of water into the natural underground reservoirs, there would be a sounder basis for the objections raised against the contentions in the paper. In discussing the question, however, several important facts have been apparently overlooked. In the first place, spreading operations are ordinarily conducted at a time when the entire stream flow would naturally be absorbed by stream channels and none wasted in the ocean. The Santa Ana spreading ground is practically the only exception, and here spreading operations are carried on only when there would be wastage of water to the ocean. It is common practice to let muddy water pass diversion gates in order to safeguard favorable conditions of spreading grounds to receive clear water.

TABLE 25.—CHANNEL ABSORPTION IN SANTA ANA RIVER*

Period	Run-off near Prado, Calif., in acre-feet†	Waste into ocean, in acre-feet	Absorption, in acre-feet
April, 1926	44 700	21 300	23 400
February, 1927	94 500	57 200	37 300
March, 1927	25 100	6 150	18 950
February, 1931	25 800	7 760	18 040

* *Water Supply Papers 631, 651*, and unpublished records, U. S. Geological Survey.

† Including run-off of Santiago Creek, near Villa Park, Calif.

This same muddy water, however, which seems to be waste from the viewpoint of the observer at the spreading grounds, forms one of the largest contributions to the ground-water from the rainfall in the mountain area. Tables 25 and 26 show that muddy water can be and is passed into ground-water.

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[78] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 198.

According to Harold Conkling,[79] M. Am. Soc. C. E., during the month of April, 1926, while 15 500 acre-ft was absorbed in the San Gabriel River channel above El Monte, only 604 acre-ft was spread at the mouth of the canyon. On the Santa Ana River more than 4 500 acre-ft of water too muddy to spread was absorbed on February 9 and 10, 1932, in a stretch of river channel not longer than 6 miles between Prado and Santa Ana, and constituted a source of ground-water supply for the Santa Ana section. This is twice as much water as the Santa Ana Water Conservation Association spread during the entire season.

TABLE 26.—CHANNEL ABSORPTION IN SAN GABRIEL RIVER*

Period	Run-off at canyon section, in acre-feet	Flow past El Monte stations, in acre-feet	Absorption, in acre-feet
March 29 to April 14, 1925	4 930	3 090	1 840
April 8 to 26, 1926	33 900	18 400	15 500
February 14 to March 11, 1927	70 400	45 400	25 000

* "San Gabriel Investigation", *Bulletin 7*, Div. of Water Rights, California Dept. of Public Works, 1929.

Apparently, the natural river channels below the spreading grounds play an important part in water conservation, and floods exercise an important function in maintaining these channels in condition to absorb water. Mr. Conkling states that,[80]

> "The rate of percolation, particularly where the stream bed contains gravel and boulders, is much less before a major flood than after such a flood. This is due to what is generally called the disruptive effect of a flood. As the water is receding, in some years at least, a thin coat of weakly cemented material is deposited in the stream channel. * * * When a flood occurs big enough to disturb the stream bed this crust is washed away and percolation rates become a maximum."

The writers do not recommend as Mr. Bates implies, the reversal of "great constructive forces of Nature." They do, however, suggest that a change should be made in the concepts to be implied by the term "water-shed protection." Heretofore, undue and in the main erroneous emphasis has been laid on "increasing the water supply," "securing favorable conditions of flow," and "promoting and protecting navigation." These concepts have been widely controverted, and it is evident that no general agreement can be reached in regard to them. The writers would emphasize their belief that water-shed protection as a means of "retarding erosion," "conserving soil," and "lessening torrential run-off," may be obtained by a cover of shrubs or grass, which may use relatively little water, as well as by trees, which may consume large quantities of water. This belief seems logical and scientifically sound and offers a platform on which engineers, foresters, and conservationists might fully agree. So considered, water-shed protection may be reasonably undertaken where physically and economically practicable and where it is clear that commensurate benefits will be obtained.

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[79] "San Gabriel Investigation," by Harold Conkling, *Bulletin 5*, Div. of Water Rights, California Dept. of Public Works, 1927.

[80] *Loc cit., Bulletin 7*, 1929, p. 56.

The Editors of the *Journal of Forestry* stated,[81]

> "The Hoyt-Troxell report has had wide circulation; from several parts of the world we learn that it has misled engineers as to the value of water-shed cover for the region in which the study was conducted."

The paper has apparently aroused interest, but it is sincerely hoped that it has not misled any one as to the value of water-shed protection. The objects of the paper will have been partly attained if those concerned with problems of water-shed protection realize that the proper solution is not solely one of silvics in relation to the head-water area, but involves a knowledge of geology, hydrology, hydraulics, and water utilization, and their relation to the entire area.

The discussion of the paper has been sufficiently complete to warrant certain definite suggestions for effective methods of increasing the water yield in the Southern California area and possibly other areas without resulting detriment. Such discussion, however, seems to be beyond the scope of the paper. Because of the limits in scope the writers have also refrained from discussing many related questions which were not covered in the original paper, but which were raised by Messrs. Sopp, Cecil, Gilman, and others. The prolonged discussion of them would cloud the principal purpose of the paper, which was to present to the Engineering Profession and to scientists in general, cumulative evidence of reliable investigations under natural conditions as to effects of trees and brush. As Mr. Randell states this evidence is to the effect that "forests and brush do not increase, but instead considerably lessen, both total water supply and low-season stream flow."

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[81] *Journal of Forestry,* Vol. 31, No. 3, p. 296, March, 1933.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1859

TESTS OF RIVETED AND WELDED STEEL COLUMNS

By Willis A. Slater,[1] and M. O. Fuller,[2]
Members, Am. Soc. C. E.

With Discussion by Messrs. Raymond J. Roark, Lyndon F. Kirkley, R. A. Caughey, E. G. Walker, M. Holt and R. G. Sturm, and M. O. Fuller

Synopsis

The tests reported in this paper were undertaken to secure a comparison of the behavior and strength of built-up steel columns fabricated by riveting, with similar columns fabricated by welding. The program included nine columns, of which two were riveted and seven were welded. In four of the welded columns, the welding was continuous throughout the length. In the other three welded columns, intermittent (or stitch) welding was used. The lengths (to the nearest inch) were 19 ft. 8 in. for three of the columns, 16 ft. 6 in. for two, 15 ft. 5 in. for three, and 5 ft. 6 in. for one.

The columns were built up by riveting or welding cover-plates to the **H**-section or the **I**-section which formed the core. The total cross-sectional area ranged from 14.26 to 24.3 sq. in. in different columns. All were tested as flat-ended columns. Observations included measurement of strains, slip of plates, deflection of column due to the applied load, and strains caused by plastic flow of the heated metal and by the stress set up when the heated metal cooled. In general, the strains were measured in various positions in cross-sections at the top, middle, and bottom of the column. These observations gave the basis for determining the bending moment distribution between the ends and the center of the column. Tension test coupons from various positions in the cross-section were used to determine the variation of yield-point stress throughout the section.

Note.—Published in September, 1932, *Proceedings*.

[1] Research Prof., Eng. Materials, and Director, Fritz Eng. Laboratory, Lehigh Univ., Bethlehem, Pa. Professor Slater died on October 5, 1931.

[2] Associate Prof., Civ. Eng., Lehigh Univ., Fritz Eng. Laboratory, Bethlehem, Pa.

A special feature was the testing of the column having an excessive initial curvature for comparison of strength, bending moment, and deflection, with those of a similar straight column.

1.—THE COLUMNS

The program herein reported, included the testing, during 1928 and 1929, of nine built-up steel columns, each consisting of a core, formed by an **I**-beam or an **H**-section, and a cover-plate attached to each flange of the core. Fig. 1 shows the details of the columns, including the methods of attaching the cover-plates to the flanges. In Columns Nos. 1, 1*BB*, 3, and 4, reinforcing plates were welded to the core section with a continuous bead. In Columns Nos. 2 and 5, plates were riveted to the core section. In Columns Nos. 6, 7, and 8, plates were welded to the core section with an intermittent bead.

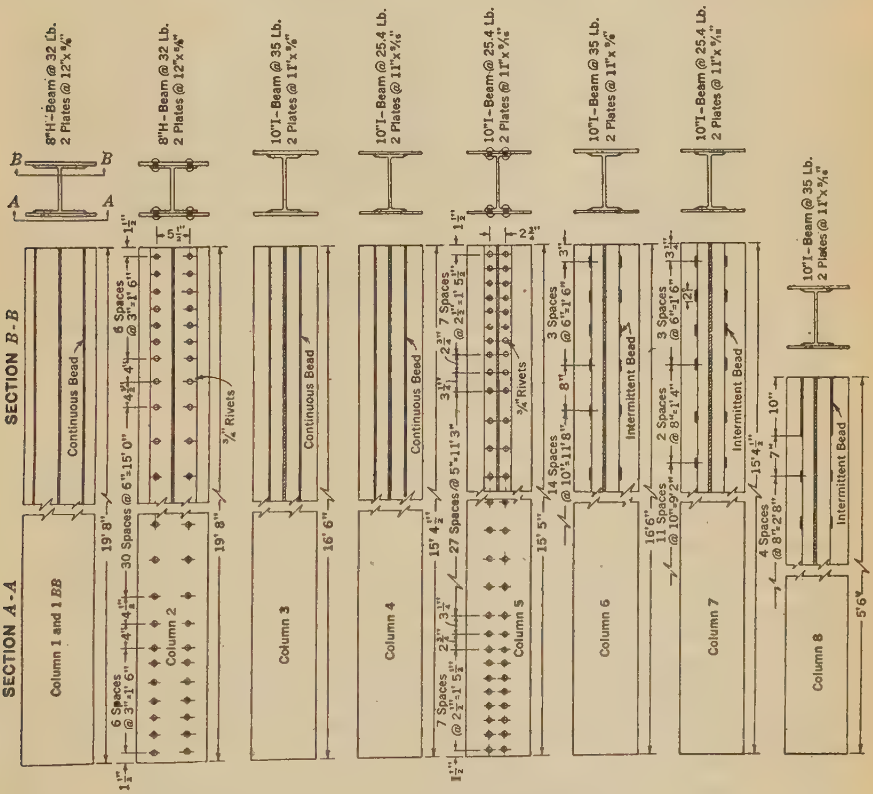
8"H-Beam @ 32 Lb.
2 Plates @ 12"x 5/8"
B
B
A
A
SECTION B-B
SECTION A-A
Continuous Bead
Column 1 and 1 BB
19' 8"
8"H-Beam @ 32 Lb.
2 Plates @ 12"x 5/8"
1 1/2"
5 1/2"
6 Spaces @ 3"=1' 6"
4 1/2"
4"
30 Spaces @ 6"=15' 0"
3/4" Rivets
Column 2
19' 8"
1 1/2"
10"I-Beam @ 35 Lb.
2 Plates @ 11"x 5/8"
Continuous Bead
Column 3
16' 6"
10"I-Beam @ 25.4 Lb.
2 Plates @ 11"x 5/16"
Continuous Bead
Column 4
15' 4 1/2"
10"I-Beam @ 25.4 Lb.
2 Plates @ 11"x 5/16"
1 1/2"
2 3/4"
7 Spaces @ 2 1/2"=1' 5 1/2"
2 3/4"
3 1/4"
27 Spaces @ 5"=11' 3"
3/4" Rivets
Column 5
15' 5"
1 1/2"
10"I-Beam @ 35 Lb.
2 Plates @ 11"x 5/8"
3"
3 Spaces @ 6"=1' 6"
8"
14 Spaces @ 10"=11' 8"
Intermittent Bead
Column 6
16' 6"
10"I-Beam @ 25.4 Lb.
2 Plates @ 11"x 5/16"
3 1/4"
2"
3 Spaces @ 6"=1' 6"
2 Spaces @ 8"=1' 4"
11 Spaces @ 10"=9' 2"
Intermittent Bead
Column 7
15' 4 1/2"
10"I-Beam @ 35 Lb.
2 Plates @ 11"x 5/8"
10"
7"
4 Spaces @ 8"=2' 8"
Intermittent Bead
Column 8
5' 6"

FIG. 1.—DETAILS OF COLUMNS TESTED.

Column No. 1 was so much out of alignment that a new column, No. 1*BB*, was furnished to take its place. However, Column No. 1 was tested in order to study the effect of its lack of alignment on its behavior under load. Previous to testing each column, its departure from a straight line in the direction of each of two planes was measured, one plane perpendicular to, and the other parallel to, the plane of the web. These departures are shown in Fig. 2. Except for differences in method of fabrication, Columns Nos. 4, 5, and 7, and Columns Nos. 3 and 6 were designed to be counterparts of columns of Types 6 and 6*A*, respectively, tested at the Bureau of Standards for the Special Committee of the American Society of Civil Engineers on Steel Columns and Struts.[3] The Special Committee provided three lengths in each type of column, and the columns here reported were originally of the intermediate length. However, in this series of tests, in order to provide material for coupons, 1 ft. was cut from the end of each column, making them 1 ft. shorter than the corresponding columns tested for the Special Committee.

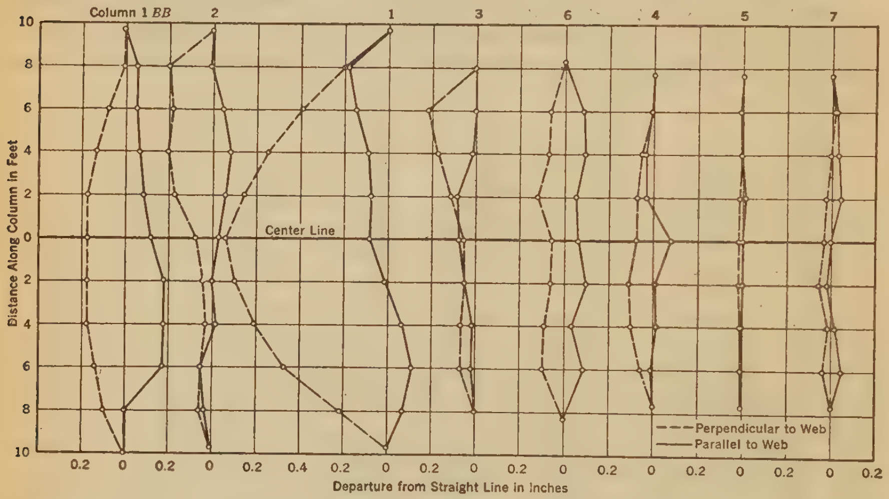

FIG. 2.—INITIAL ALIGNMENT OF COLUMNS.

The columns were tested with "flat ends" as nearly as this condition could be approached. The top and bottom of each column had been milled, but the milling did not give surfaces as nearly plane as might be desired. It is believed that lack of planeness of the ends affected somewhat, the location of the load line in the columns as the loads were applied. Details of methods of testing are given in the Appendix. The data of the test specimens are given in Table 1.

Nomenclature.—The term, "strain line," introduced in this paper, is used to designate the markings that appeared in the white cement coat (which had been applied to all the columns prior to testing) when the mill scale flaked off due apparently to the condition of stress in the steel. The expression, "unit deformation," and the word, "strain," are used to mean the change of length per unit of length, and the word, "stress," to mean the intensity of the internal force per unit of sectional area.[4] The expression, "unit deformation," is applied to a change of length due to any cause such as stress, or expansion due to heat, while the word, "strain," should be applied only to a change of length caused by stress. However, it is not always possible to determine what is the principal cause of a change of length and there may be some confusion in the use of the two terms. In the following notes the term "inside" or "interior" face of a cover-plate or of a flange, refers to the face toward the center of the column. Similarly, "outside" or "exterior" face refers to the face away from the center of the column.

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[3] *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1598.

TABLE 1.—DATA OF TEST COLUMNS

Column No.	Type	Length, in inches	Core	PLATES		Sectional area of column, in square inches	I, in inches^4	r, in inches	$\frac{l}{r}$	Average stress at maximum load, in kips* per square inch
				Width, in inches	Thickness, in inches					
1	Continuous weld....	236	8-in. H, 32 -lb.	12	$\frac{5}{8}$	24.30	206.6	2.92	80.8	32.3
1*BB*	Continuous weld....	236	8-in. H, 32 -lb.	12	$\frac{5}{8}$	24.30	206.6	2.92	80.8	32.3
2	Riveted...	236	8-in. H, 32 -lb.	12	$\frac{5}{8}$	24.30	206.6	2.92	80.8	32.5
3	Continuous weld....	198	10-in. I, 35 -lb.	11	$\frac{5}{8}$	23.97	155.0	2.54	77.9	30.8
4	Continuous weld....	184.5	10-in. I, 25.4-lb.	11	$\frac{5}{16}$	14.26	78.6	2.35	78.7	36.9
5	Riveted...	185	10-in. I, 25.4-lb.	11	$\frac{5}{16}$	14.26	78.6	2.35	78.7	35.8
6	Stitch weld	198	10-in. I, 35 -lb.	11	$\frac{5}{8}$	23.97	155.0	2.54	77.9	33.3
7	Stitch weld	184.5	10-in. I, 25.4-lb.	11	$\frac{5}{16}$	14.26	78.6	2.35	78.7	36.7
8	Stitch weld	65	10-in. I, 35 -lb.	11	$\frac{5}{16}$	17.10	77.8	2.12	31.1	†

* One kip equals 1 000 lb. †Not tested to failure.

2.—PHENOMENA OF TESTS

The visual phenomena of the tests consisted principally of the formation of the "strain lines" in the whitewash caused by the buckling or flaking of the mill scale. Their occurrence is described in considerable detail for Column No. 4 which may be considered as representative. The significance of the strain lines in general is discussed in Section 4.

Strain lines first appeared in Column No. 4 (welded) at an applied load of 266 kips (approximately 19 kips per sq. in.). They appeared along lines extending horizontally from the edges toward the centers of the cover-plates for a distance of about 2 in. None was appreciably longer than 2 in. Strain lines were first observed near the top after which they extended rapidly over the entire length of the column. They occurred in pairs, one appearing on the outside face of the cover-plate for each one on the inside face. The vertical distances between the strain lines on the outside and inside faces were seldom more than $\frac{1}{8}$ in. With the machine stopped at a load of 266 kips, the number of strain lines continued to increase, but did not extend beyond 2 in. from the edge of the plate, nor did any appear on the I-section. A few forked slightly near their ends.

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[4] *Proceedings*, Am. Soc. for Testing Materials, Vol. 25 (1925), Pt. I, p. 897.

Under an applied load of 308 kips (approximately 22 kips per sq. in.) strain lines appeared in the I-section, extending from the fillets outward toward the edges of the flanges (Fig. 3). These strain lines at first were about $\frac{3}{8}$ in. long. As the load increased they extended slightly, but never reached a length greater than $1\frac{1}{4}$ in., and did not extend into the web. Some of them were inclined, but generally they were horizontal. At the same load (308 kips) strain lines were found on the outside of the cover-plates crossing the plane of the web at the top within about 3 in. of the end of the column. As the loading continued, these strain lines extended and became more pronounced. Those that had developed earlier in the test showed more forking at the ends.

The distribution of strain lines after the completion of the test is well shown in Figs. 3 and 5. The flanking of the whitewash merely accentuated the phenomenon of the separation of the mill scale from the steel. At places, patches of mill scale of considerable size separated from the metal. In Fig. 3 the shapes of the strain lines are shown for the web and the inner faces of the flange. The uniformity with which the strain lines in the web of the columns extended, about 2.5 in. on either side of the vertical center line of the column, is notable, as well as the regularity with which the strain lines in the flanges began at the fillet and terminated at a vertical line about 1.5 in. from the fillet.

The location of strain lines on the exterior face of the cover-plates, corresponds closely with that of the strain lines on the interior face of the cover-plate and the flange of the I-beam. Most of the strain lines were horizontal at the time of reaching the maximum load. After reaching the maximum load the testing machine was kept running until the load had decreased to about 500 kips. The photographs were taken after the completion of the test, and figures formed by the flaking show many more diagonal lines and much more forking than were present at the maximum load. The vertical line that appears in Fig. 5 at about the position of the web of the column is not due to strain in the plate, but to accidental abrasion of the whitewash. There were no strain lines at the junction of the plate with the I-section. At the higher loads the column deflected markedly in a direction at right angles with the web. No other marked phenomenon of failure appeared.

With most of the columns, the cause of failure appeared to be a deflection over the entire length in a plane at right angles with the plane of the web. In Columns Nos. 5 and 7, however, local buckling of the cover-plates between points of attachment took place at the maximum load, and this apparently was the immediate cause of failure. Both these columns had 11 by $\frac{5}{16}$-in. cover-plates.

In Column No. 5 the buckling occurred on only one side, at about mid-height where the rivets were 5 in. apart in the vertical direction. It had been noticed at the beginning of the test, before any load was applied, that the plate which buckled was slightly bent at the bottom bearing, but there was nothing to indicate that this slight crippling had affected the behavior of the column. The average stress on the entire section was approximately 35.8 kips per sq. in. at the time that the buckling occurred.

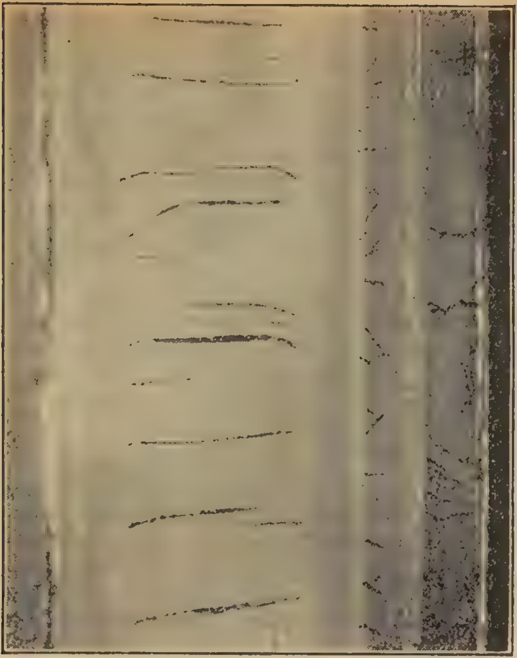

Fig. 3.—Strain Lines on Web of Column No. 4.

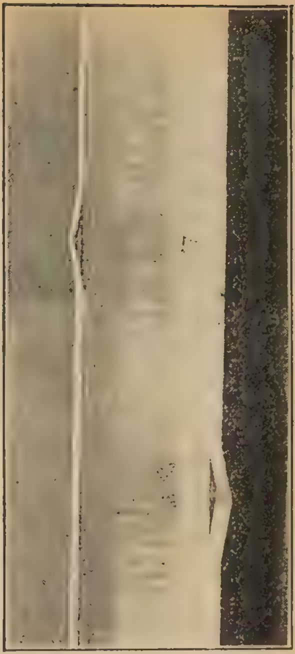

Fig. 4.—Column No. 7 After Test Showing Strain Lines and Buckling of Cover-Plates.

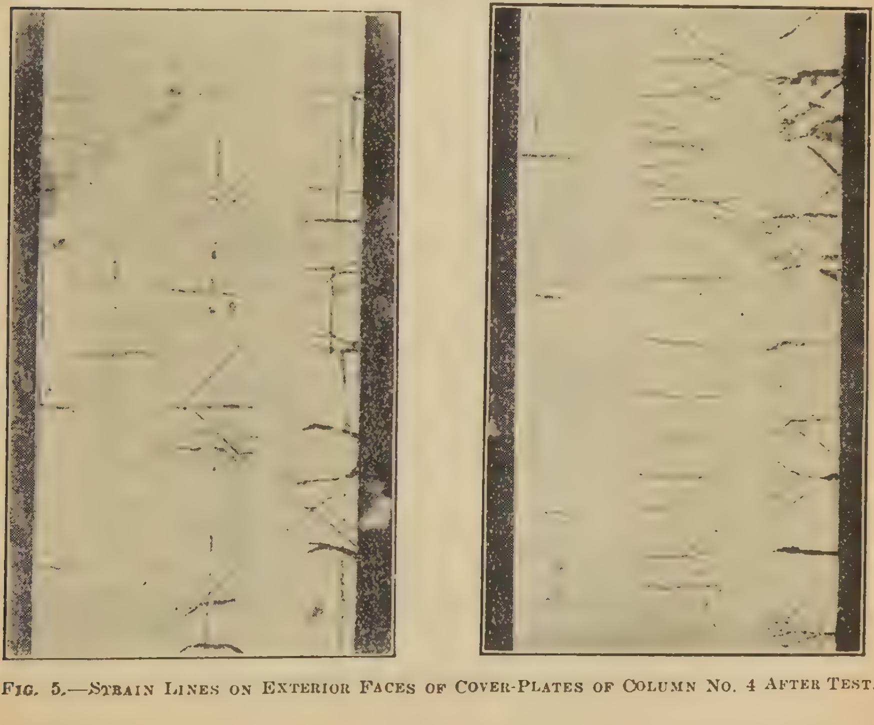

Fig. 5.—Strain Lines on Exterior Faces of Cover-Plates of Column No. 4 After Test.

In Column No. 7 the buckling occurred in both cover-plates, one at 10 in. and the other at 31 in. below mid-height, as shown in Fig. 4. In this column the distance from center to center between the 2-in. stitch-welds, was 10 in. No defects in the welds at the points of buckling were observed. Buckling of the plates occurred in both the riveted and the stitch-welded columns with $\frac{5}{16}$-in. cover-plates, but did not occur in either the riveted or the stitch-welded column with $\frac{5}{8}$-in. cover-plates. The ratios of the rivet or weld spacing to the thickness of plates are given in Table 2.

TABLE 2.—RATIOS OF RIVET OR WELD SPACING TO THICKNESS OF PLATES

Column No.	Type	Plate thickness, t, in inches	Center to center spacing, s, in inches	$\frac{s}{t}$	Remarks
2	Riveted	$\frac{5}{8}$	6	9.6	Plates did not buckle
5	Riveted	$\frac{5}{16}$	5	16	One plate buckled
6	Stitch-welded	$\frac{5}{8}$	10	16	Plates did not buckle
7	Stitch-welded	$\frac{5}{16}$	10	32	Both plates buckled

3.—SLIPPING OF PLATES

Gauge Lines $B5$, $B11$, $B17$, and $B23$ (Fig. 6), were placed so that their changes in length indicated the movement of a point on the cover-plate rela-

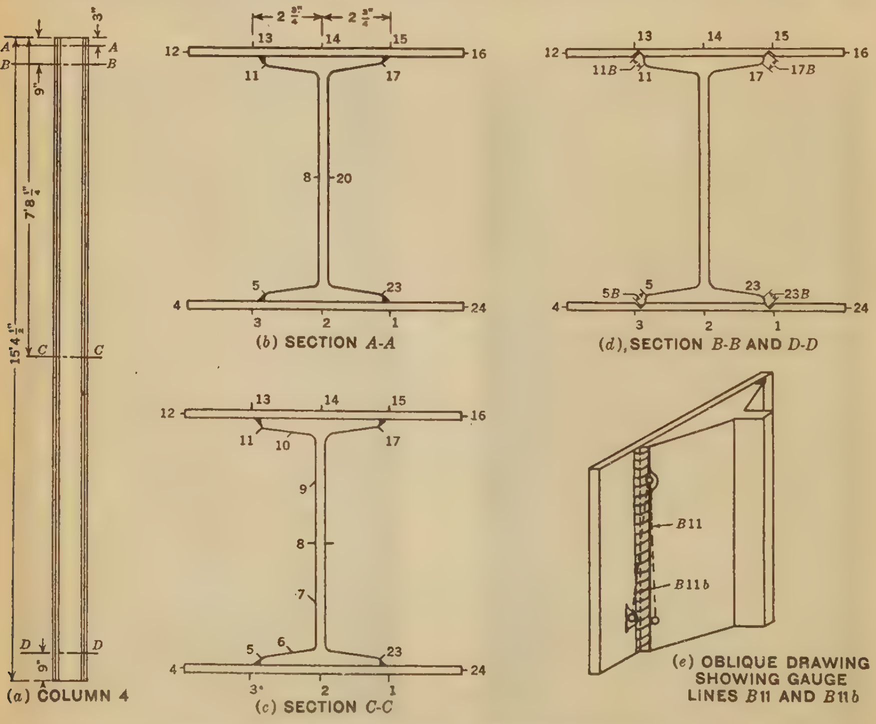
3"
A
B
9"
7' 8¼"
15' 4½"
C
D
9"
(a) COLUMN 4
2¾"
2¾"
13
14
15
12
16
11
17
8
20
5
23
4
24
3
2
1
(b) SECTION A-A
11B
17B
5B
23B
(d), SECTION B-B AND D-D
10
9
7
6
(c) SECTION C-C
B11
B11b
(e) OBLIQUE DRAWING SHOWING GAUGE LINES B11 AND B11b

FIG. 6.—LOCATION OF GAUGE LINES, COLUMN NO. 4.

tive to a point on the **I**-section or **H**-section. These gauge lines were placed only on Section *B* at the top of Column No. 4, and on both Sections *B* and *D* for Columns Nos. 1*BB*, 2, 3, 5, 6, and 7, since it was believed that if slipping occurred at all, it would be near one end of the column. Gauge Lines *B*11 and *B*11*b* are shown in Fig. 6(*e*). Gauge Lines *B*5, *B*5*b*, *B*17, *B*17*b*, *B*23, and *B*23*b*, are similarly arranged. Fig. 7 presents some representative curves showing the movement in various gauge lines. They have been plotted in pairs, both gauge lines of each pair being referred to the same origin. The dotted lines are the graphs for Gauge Lines 5, 11, 17, and 23, and the solid lines are for Gauge Lines 5*b*, 11*b*, 17*b*, and 23*b*. The numbers designating the gauge lines are shown near the origin of the respective graphs. The capital letters, *B* or *D*, indicate whether the measurement was at Section *B* or Section *D*, and the Arabic numerals indicate the position of the gauge line in the section. (See Fig. 6.) Similar sets of curves were drawn for the other columns.

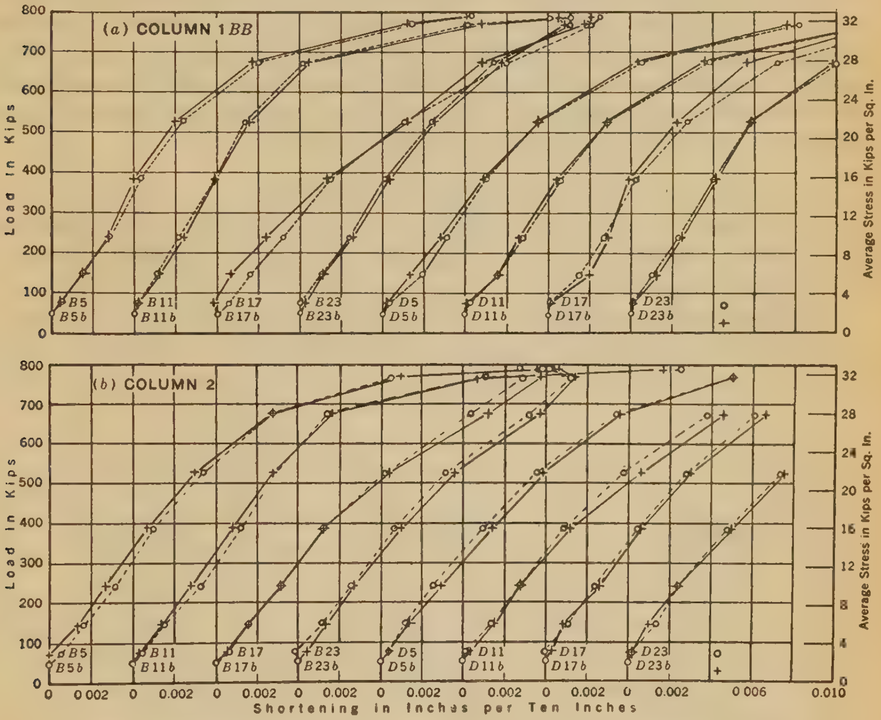

FIG. 7.—STRESS-STRAIN CURVES FOR DETERMINATION OF SLIP OF PLATES.

If there were no slipping of the plates, and no error of observation, the dotted lines and the solid lines should coincide. Any divergence indicates either error of observation or slipping of the plates, or both. Since there is certain to be some error of observation, some divergence of the lines is inevitable. If an error occurs in the zero reading, it will cause the dotted and the solid graphs to remain at a fixed horizontal distance apart; if it occurs in readings for loads above that for the zero reading, it will cause an erratic divergence and convergence of the graphs. Slipping of the plate with reference to the structural shape should be expected to increase with increased load, and, therefore, should cause a progressive divergence of the graphs.

The curves, including those in Fig. 7, show many erratic divergences and convergences of the lines, most of which are slight, indicating slight errors in the measurements. There are a few slight divergences which are progressive over a part of the test, such as that for Gauge Lines *D*17 and *D*17*b*, for Column No. 1*BB*, and that for Gauge Lines *D*11 and *D*11*b*, for Column No. 2 (Fig. 7). These divergences, however, represent at most a slip of approximately 0.001 in., and it cannot be stated positively that it represents a slip at all. On the whole, the practical coincidence of the dotted and the solid graphs in these diagrams indicate that there was little if any slipping of the plates in either the welded or the riveted columns.

4.—Strains and Strain Lines

Fig. 6 shows the positions of the gauge lines. While the cross-sections of the different columns varied, the same plan of gauge numbering was used throughout the series.

Bending strains developed quite early in the test, although the bearings were adjusted in such a way that, at an applied load of approximately 60 kips, the observed strains at the four corners of the column were equal. It was assumed that the over-all shortening, which was used in making the adjustments of the bearing (see Appendix, Article 2), would detect bending strains in the columns due to eccentricity of the bearing. However, an examination of Column No. 4 (see Fig. 8), indicated bending moments at the middle and

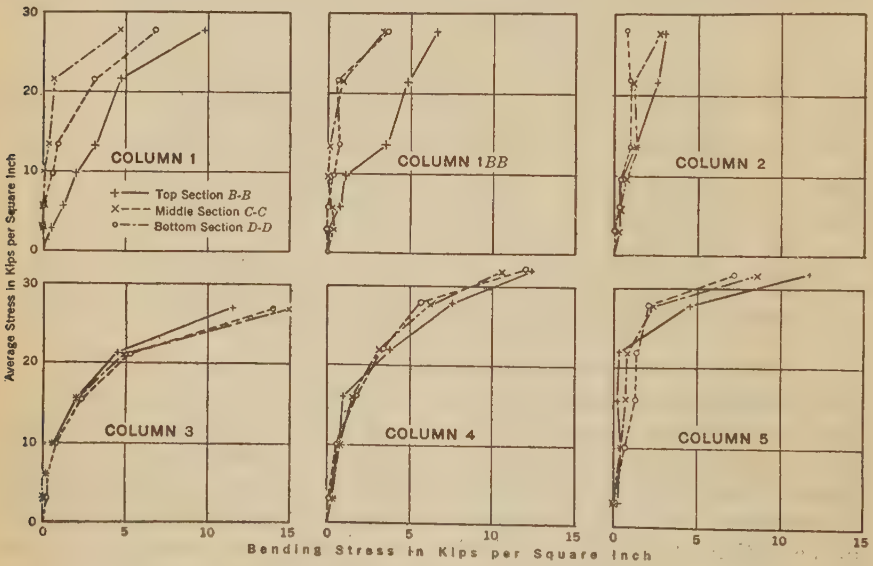

Fig. 8.—Bending Stresses in Columns.

at the ends nearly equal in value and opposite in sign. This reversal of moment tended to equalize the over-all shortening measured on the four corners of the column, and, therefore, reduced the effectiveness of these measurements as an adjusting agent. It is evident that if the over-all shortening at the four corners of the column were equal in spite of large bending moments in the column, the measurement of this shortening would be of no use in detecting eccentricity of load.

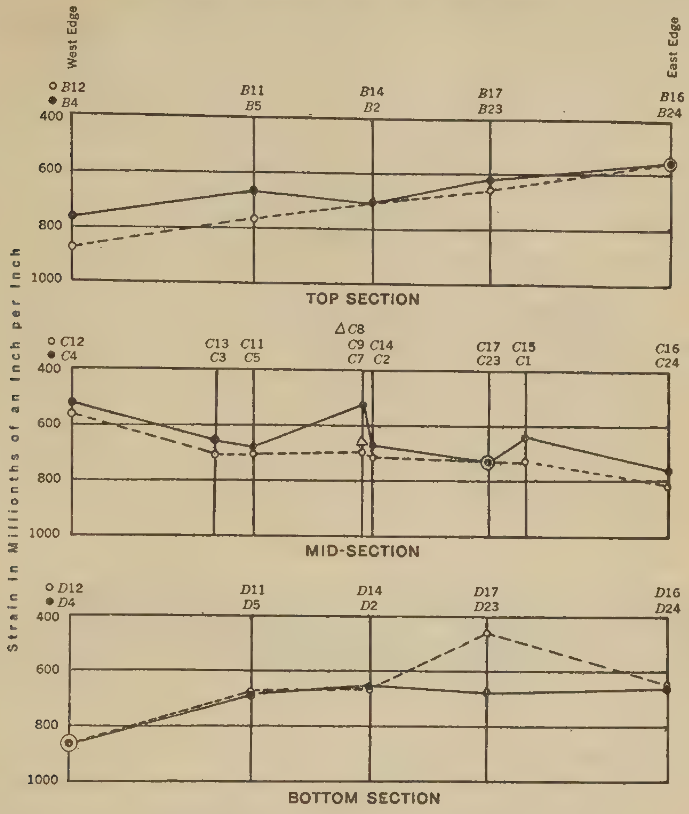

FIG. 9.—DISTRIBUTION OF STRAINS AT SECTIONS OF COLUMN NO. 4 FOR A LOAD OF 308 KIPS, OR A STRESS OF 216 KIPS PER SQUARE INCH.

The presence of bending strains is indicated by the slope of the graphs in Fig. 9, which shows the total measured strains for the cross-sections at *BB*, *CC*, and *DD* of Column No. 4. Fig. 10 shows a pronounced change of slope of the average stress-strain curve at the stress of 21.6 kips per sq. in. Fig. 9 is more or less typical of the strain distribution across the sections of the columns for loads near the critical.

Up to a certain load for each column, the average strain at mid-section was approximately the same as the average for the full length of the column. Above this load the average strain at the center was greater than the average for the full length of the column. This is shown by the divergence of the broken from the solid lines, for the higher loads in Fig. 10.

Figs. 3, 4, 5, 11, 12, 13, 14, and 15 are representative views of the strain lines that appeared in columns of various types and designs during the testing. They were all taken after the completion of the individual tests. Evidently, two major influences affected the number, location, and direction of the strain lines in the whitewash, namely, the weight of the section, and the method of fabrication. In the column of the lighter sections, the strain lines were much more general than with the heavier sections. With the 10-in., 25.4-lb. **I**-sections, the strain lines in the web were closely spaced and prominent, but never extended into the fillets, either from the webs or from the flanges. (See Figs. 3 and 4.) With both the heavier cores; that is, the 8-in., 32-lb. **H**-sections (Fig. 14) and the 10-in., 35-lb. **I**-sections, the strain lines were less general, but they extended entirely across the web and out into the flanges.

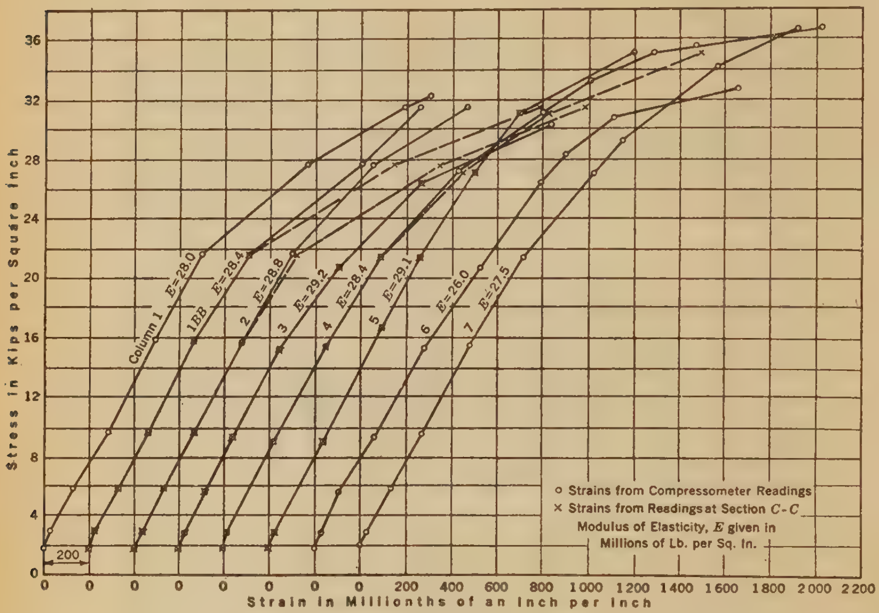

FIG. 10.—AVERAGE STRESS-STRAIN CURVES FOR COLUMNS NOS. 1 TO 7.

No case was found in which a strain line crossed a weld, and only a few in which they crossed the line of the weld on the outer face of the plate; that is, on the face opposite that of the weld (Fig. 15). In the stitch-welded columns (Fig. 4), the same general placement of the strain lines existed as in the columns with the continuous welds, except that in the former the strain lines, generally, were found in groups opposite the welds and seldom in the sections between the intermittent welds. With the riveted columns, the strain lines extended to the edge of the flange and up close to the rivets. (See Figs. 11 and 12.)

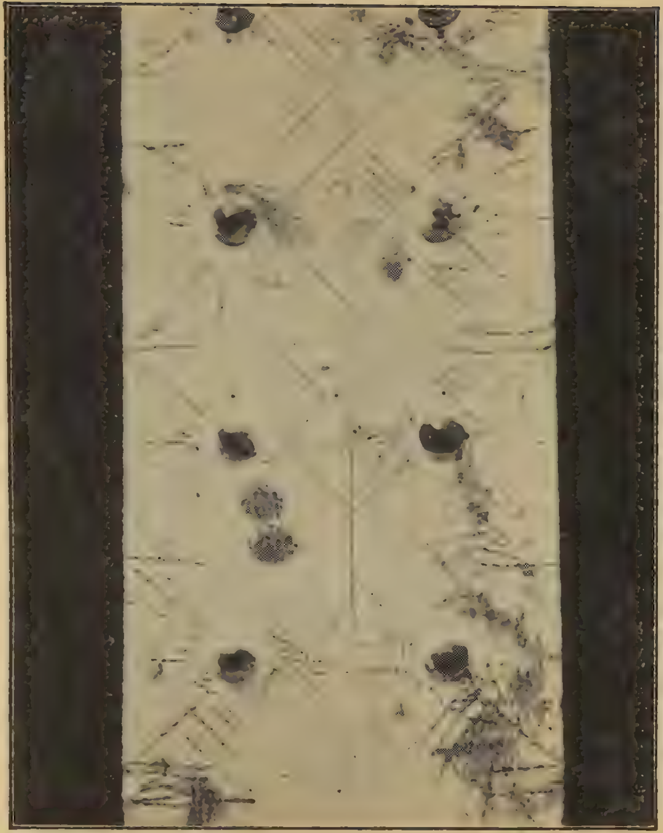

Fig. 11.—Strain Lines on Exterior Face of Cover-Plate of Column No. 2 After Test.

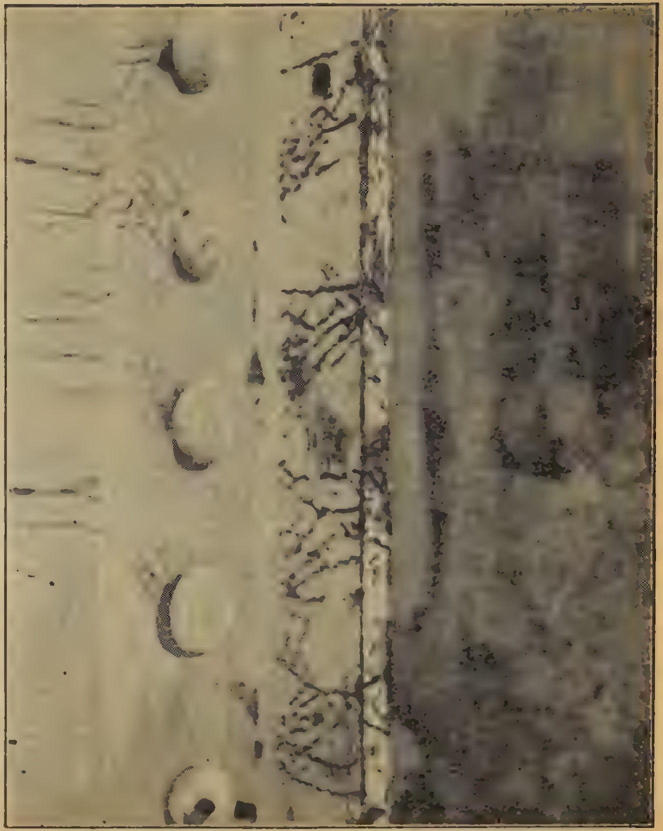

Fig. 12.—Strain Lines on Interior Face of Cover-Plate of Column No. 2 After Test.

In each of the columns the first strain line appeared in one of the cover-plates, generally near the edge, at the stress given in Table 3, Column No. 4. The stress at this time ranged from 21 to 10 kips per sq. in., except for Column No. 6, in which the first strain lines appeared at a stress of 5.8 kips per sq. in. The average stress at which a strain line first appeared in the cover-plates was 15.1 kips per sq. in. The average stress when a strain line first appeared in the web is shown in Table 3 to have ranged from 36 to 22 kips per sq. in. for the various columns.

TABLE 3.—COMPARISON OF COMPUTED STRESSES FOR MAXIMUM LOAD ON COLUMN WITH YIELD POINT STRESSES FROM COUPONS AND WITH STRESSES COMPUTED BY RANKINE'S FORMULA

Column No.	Yield point of coupons, in kips per square inch	STRESS AT FIRST STRAIN LINE, IN KIPS PER SQUARE INCH		Stress at proportional limit of column, in kips per square inch	STRESS AT MAXIMUM LOAD, IN KIPS PER SQUARE INCH		Ratio of stress at maximum load to yield point of coupons	Ratio of stress from Rankine formula to stress at maximum load
		Cover-plate	Web		From test	Computed by Rankine formula		
1	34.9	16.0	22.0	21.8	32.3	27.7	0.925	0.858
1*BB*	35.4	16.0	28.0	21.8	32.3	28.1	0.912	0.870
2	35.9	16.0	32.0	21.8	32.5	28.5	0.905	0.877
3	34.6	21.0	27.0	15.3	30.8	27.8	0.892	0.903
4	38.7	19.0	29.0	36.9	36.9	31.0	0.954	0.840
5	39.2	17.0	36.0	21.4	35.8	31.4	0.913	0.877
6	37.9	5.8	24.0	15.4	33.3	30.5	0.879	0.916
7	39.8	10.0	27.0	15.4	36.7	31.9	0.922	0.869

The stress at which strain lines appear is generally assumed to be approximately the yield point of the steel. For this to have been the case with these columns, requires that there must have been an initial stress at the point where the first strain lines started, averaging about 20 kips per sq. in. There are some reasons for thinking that compressive stresses were set up in the webs of the **I**-beams and **H**-beams by unequal cooling after rolling, but it is not possible from the data of these tests, to estimate their magnitude. Although initial stresses of considerable magnitude due to the heat of rolling and of welding were undoubtedly present, this fact need not be alarming since the strengths of the columns were about what should be expected with the materials, shapes, and lengths used.

5.—EFFECT OF WELDING ON STRAINS AND STRESSES

In a study of the strains due to welding, and their effect on the location of strain lines, Column No. 8 (5 ft. 6 in. long), was fabricated and tested. (See Fig. 13.) Two, 11 by $\frac{5}{16}$-in. cover-plates were stitch-welded to a core that consisted of a 10-in. 35-lb. **I**-beam. The welds at the top and bottom of the column were continuous for 10 in. from the ends. The intermediate welds were each 2 in. long and spaced 8 in. apart, center to center. Previous to the welding of the plates to the **I**-section, numerous 2-in. gauge lines were established and read. These lines were read again after the welding was completed and after the column had cooled down to normal temperature. Load was then

FIG. 13.—STRAIN LINES ON EXTERIOR FACE OF COVER-PLATE OF COLUMN NO. 8 AFTER LOADING TO AVERAGE STRESS OF 27.7 KIPS PER SQ. IN.

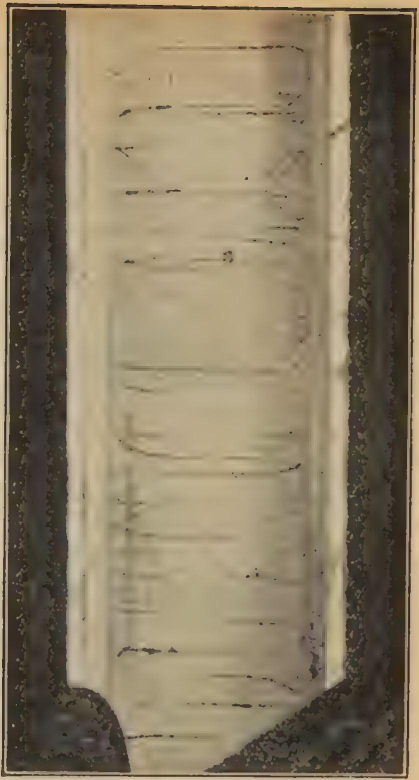

FIG. 14.—STRAIN LINES ON WEB OF COLUMN NO. 1*BB* AFTER TEST.

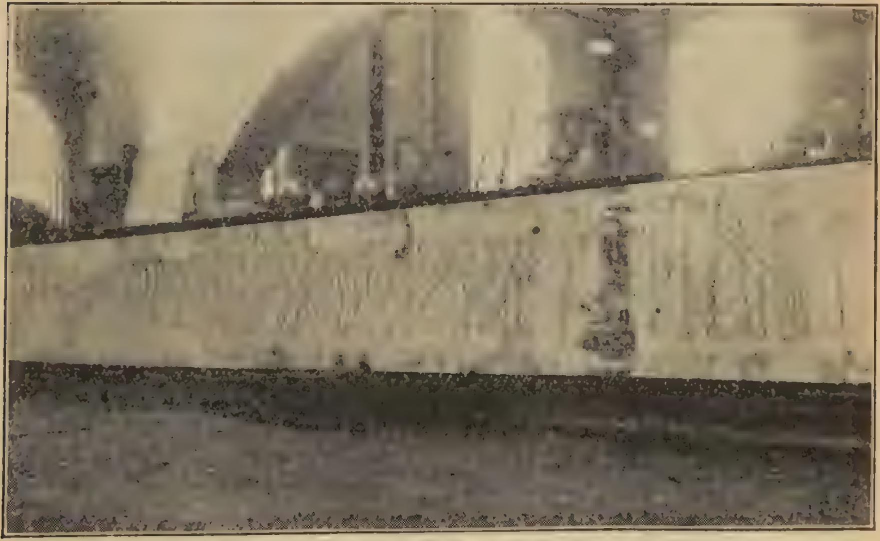

FIG. 15.—STRAIN LINES ON EXTERIOR FACE OF COVER-PLATE OF COLUMN NO. 1 AT MAXIMUM LOAD.

applied up to a maximum of 398 kips, which caused an average stress in the column of 27.7 kips per sq. in. After the strain readings had been obtained this load was removed and another set of readings was taken to measure any permanent set in the column.

Fig 16 shows the location of intermittent welds in relation to the gauge lines used to determine the distribution of the strains in a cross-section at the center of the column and along the edges of the cover-plate for some distance above and below the center. The elevation of the gauge lines is given by letters, and the position on the cross-section, by numbers. For example, *D*-25 was at the mid-height of the column and on the fillet of the I-section.

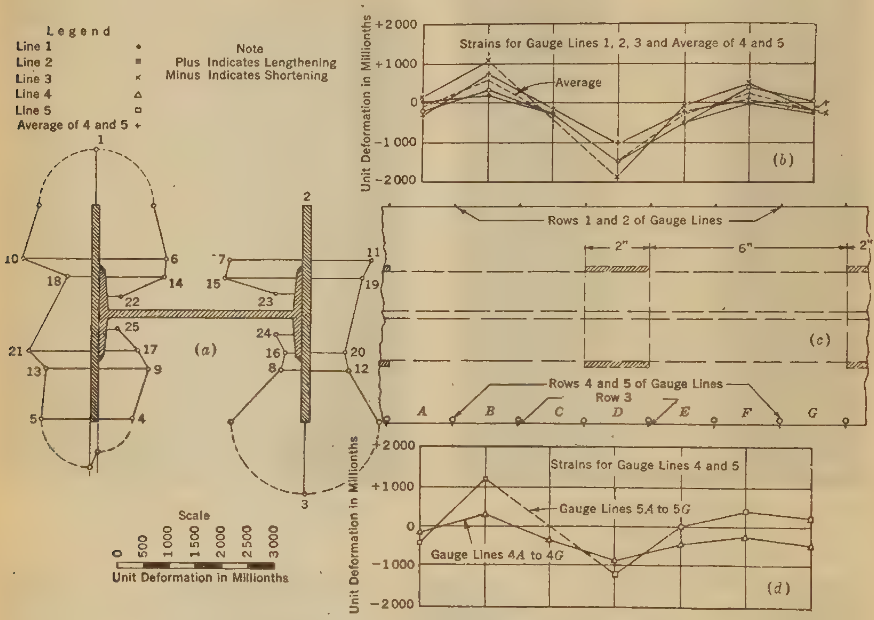

FIG. 16.—STRAINS IN COLUMN NO. 8 DUE TO WELDING.

In Fig. 16 (*a*), the unit deformations in all gauge lines cut by Section *D* at mid-height of the column have been plotted opposite the gauge line in a direction perpendicular to the surface in which the strain occurred. A distance away from the surface represents unit shortening. Evidently, all the gauge lines cut by the section were shortened, although it is unlikely that the stress was compression in all of them, since the total stress on the section was zero. There were no gauge lines on the web, but it does not seem likely that the web was in tension due to bending stresses.

Without some knowledge of how much plastic flow there was, it is impossible to state how many of the resultant deformations shown in Fig. 16 (*a*) represent stress. However, at points in the section so far removed from the weld that plastic flow due to the heat was unlikely, the strains indicate a compressive stress of about 15 kips per sq. in. and it seems likely that at the weld the tensile stress was as great as the yield point of the metal, in spite of the fact that the strain readings showed that it had shortened due to the welding.

Fig. 16 (*b*) shows that at all four corners of the column there was a marked shortening at the section through the welds and a marked elongation at the sections between welds. Sections *B* and *F*, midway between welds, were far enough from the weld so that probably no plastic flow occurred in them. The average tensile strains at the edges of the plates for these two sections were 600 millionths and 250 millionths, respectively. Using the average modulus of elasticity of 29 000 000 lb. per sq. in. the corresponding stresses are 17.4 and 7.25 kips per sq. in. Since the external force on each of these sections is zero, there must have been compressive stresses on other parts of the section to balance the tensile stresses.

The variations of strain along the edges of the plate, as shown in Fig. 16 (*b*), make evident why the strain lines on the welded columns appeared first opposite the welds. Evidently, the heat of welding had set up compressive stresses at these points and tensile stresses at the edges of the cover-plates between them, so that when the load came on the column, the yield point was reached first at points opposite the weld.

Fig. 16 (*d*) shows that the strains on opposite faces of the cover-plate were not equal, but that local bending strains, varying somewhat regularly from weld to weld, took place. These bending strains were probably due to the fact that the heat was applied to only one side of the plate. The difference in strains corresponds to a difference of stress on opposite sides of the plate, amounting at Section *B* to approximately 27, and, at Section *F*, to 19.5 kips per sq. in.

The phenomena pointed out in the foregoing paragraphs make it apparent that there is room for much further study of the effect of welding on stresses in structural members.

Upon application of load to the column, the strains were read and have been plotted in Fig. 17, in which the letters, *B*, *D*, and *F*, refer to positions shown in Fig. 16. The slope of the graphs indicates that the strains increased at approximately the rate to be expected for steel under stress, but beginning in each case with the strain set up by the welding. As a result, the total strains at Gauge Lines *D*1 to *D*21, under the final stress of 27.7 kips per sq. in., were very great. Strain lines formed in the white cement coat at the indicated points of high strain opposite to, and at some distance from, the welds, but none close to the welds. This is consistent with the fact that, as was previously pointed out, tensile stresses must have existed in the immediate vicinity of the welds due to shrinkage on cooling. The absence of strain lines in the welds may have a further explanation since auxiliary tests made on small angles showed no strain lines on the welds, although the load applied caused stresses far beyond the yield point of the steel.

6.—MODULUS OF ELASTICITY

Table 4 gives the moduli of elasticity as determined for certain coupons and for the columns. The moduli of elasticity of the coupons for the core sections and the plates were weighted in proportion to the sectional areas of each, and the weighted average for the entire cross-section of the column was computed. To determine the moduli directly from the tests of the columns the strains in the gauge at mid-height (Section *CC*) were weighted in proportion to the approximate areas of the cross-sections which they represented, and weighted average strains for Section *CC* were computed from these weighted values. These strains were plotted in Fig. 10, and from these stress-strain diagrams the moduli of elasticity, given in the last column of Table 4, were computed.

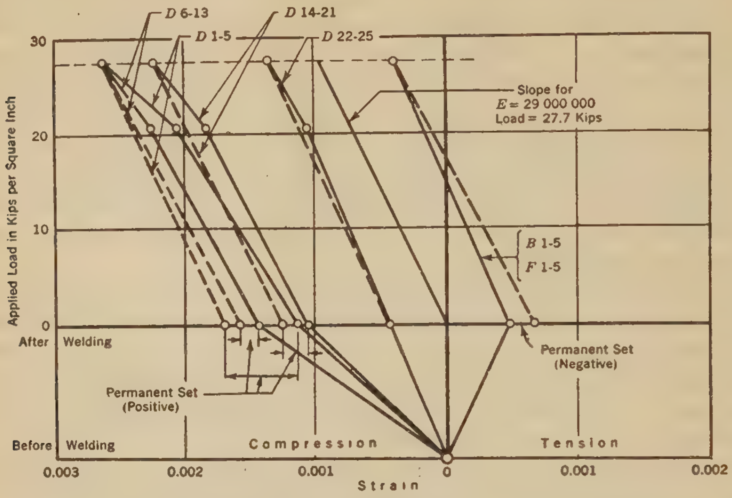

FIG. 17.—LOCAL STRAINS IN COLUMN NO. 8 BASED ON READINGS TAKEN BEFORE FABRICATION.

TABLE 4.—MODULUS OF ELASTICITY OF COLUMNS, IN POUNDS PER SQUARE INCH

Column No.	MODULUS FROM COUPON TESTS			From column tests
	Core section	Plates	Weighted average	
1	29 500 000†	30 000 000†	29 800 000	28 000 000
1*BB*	28 400 000†	30 100 000†	29 500 000	28 400 000
2	31 000 000†	30 400 000†	30 600 000	28 800 000
3	31 300 000†	28 900 000†	29 900 000	29 200 000
4	28 000 000†	27 400 000†	27 700 000	28 400 000
5	27 500 000†	29 100 000*	28 300 000	29 100 000
Average....	29 300 000	29 300 000	29 300 000	28 650 000

* Average for four test coupons. † Average for two test coupons.

Only the first five columns tested, Nos. 1*BB*, 2, 3, 4, and 5, were used for these determinations, since in Columns Nos. 1, 6, 7, and 8, some or all of the gauge lines on Section *CC* were omitted. The extreme range in moduli of elasticity from the coupons tested in tension, was about 10% of the average value. The corresponding range of values from the column tests (compression) was about 3 per cent.

The average modulus from the coupons was 29 300 000 and that from the columns, 28 650 000 lb. per sq. in. The agreement between the two values is reasonably good, and their average is very close to 29 000 000 lb. per sq. in., the value used in the computation of stresses from strains in this paper.

The agreement between the weighted moduli for the columns and the coupons is of interest in view of the fact that tests of two independent structures in recent years reported by Clyde T. Morris,[5] M. Am. Soc. C. E., and A. H. Fuller,[6] M. Am. Soc. C. E., indicated that either (1) the modulus of elasticity of the columns was only about 22 300 000 lb. per sq. in.; (2) the load was about 27% greater than that computed; or (3) errors due to temperature deformations entered the data. Both tests appear to have been conducted so carefully, and the results were so consistent for a large number of columns, as to make it unlikely that appreciable error due to temperature changes or inaccuracy in determining loads was involved.

It was largely in an effort to find out whether uneven distribution of stress over the cross-section could explain the discrepancy in the tests by Professors Morris and Fuller that the gauge lines shown in Fig. 6 were distributed so completely over the cross-section of the column. However, the results of these tests offer no explanation of the unexpected phenomena observed by Professors Morris and Fuller.

7.—Deflections

The initial alignments of the columns as given by measurements on the cover-plates, are shown in Fig. 2. The load-deflection curves are given in Fig. 18, and the deflections at maximum load in Table 5. In both cases the deflection given occurred in the plane of least resistance.

TABLE 5.—Results of Column Tests

Column No.	Sectional area, in square inches	$\frac{l}{r}$	Maximum Load: Total In kips	Maximum Load: per Unit Area In kips per square inch	Deflection* at maximum load, in inches	Modulus of elasticity, in pounds per square inch
1	24.30	80.8	785	32.3	0.68	28 000 000
1*BB*	24.30	80.8	784	32.3	0.33	28 400 000
2	24.30	80.0	790	32.5	0.50	28 800 000
3	23.97	77.9	739	30.8	0.75	29 200 000
4	14.26	78.7	527	36.9	0.77	28 400 000
5	14.26	78.7	510	35.8	0.87	29 100 000
6	23.97	77.9	799	33.3	0.62	26 000 000
7	14.26	78.7	523	36.7	0.38	27 500 000
Average.....	.....	....	...	33.8	0.61	28 155 000

* Deflection measured perpendicular to web.

The following discussion of deflections applies to Column No. 4 which was the first column tested. The behavior indicated here may be taken as more or less typical of all except Column No. 1 which is discussed in Section 8 and except Column No. 8 for which no deflections were measured. Column No. 4 was not entirely straight at the beginning of the test. (See Fig. 2.) Each cover-plate had a camber of 0.08 in. at the center in the plane of the plate, but the camber was in opposite directions for the two plates, so that for deflections of the column as a whole, the effect should be nearly the same as if the column had been straight. The camber in opposite directions might tend to introduce a twisting, but if any twist was present it was not large enough to be observed visually.

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[5] "Dead Load Stresses in the Columns of a Tall Building," *Bulletin No. 40*, Eng. Experiment Station, Ohio State Univ.

[6] "Measurement of Stresses in Four Steel Columns of the Equitable Building," *Bulletin No. 72*, Eng. Experiment Station, Iowa State Coll.

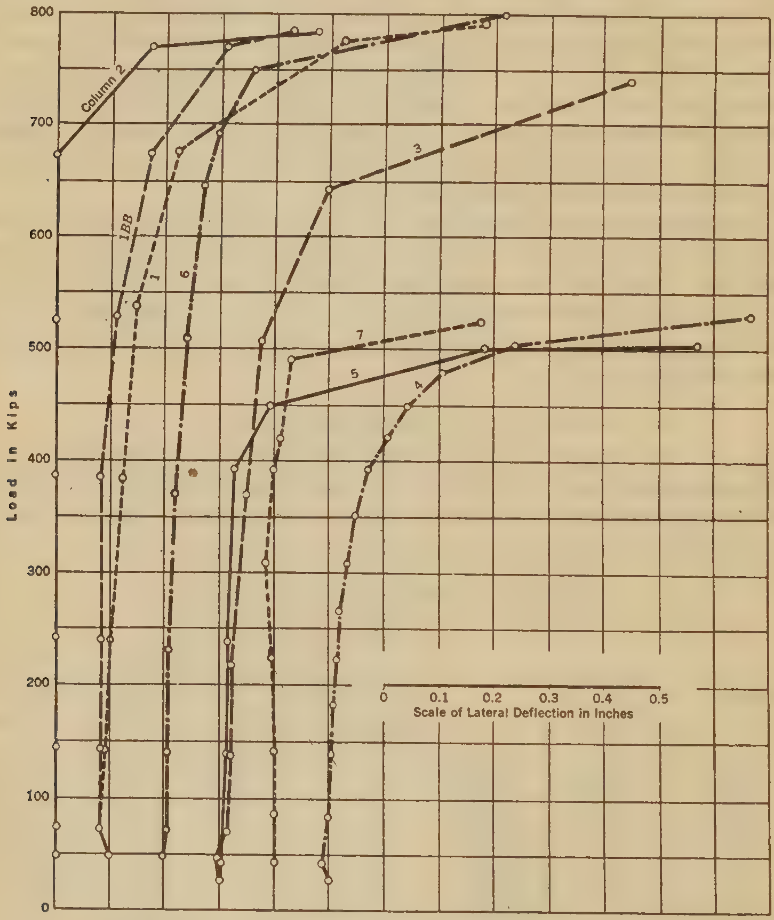

FIG. 18.—CENTER DEFLECTION PERPENDICULAR TO WEB.

Some deflection in the plane of least resistance was apparent almost from the beginning of the test (Fig. 18). At a load of 266 kips, this deflection was 0.02 in. and from that point the deflection increased rapidly, becoming 0.77 in. at the maximum load of 527 kips. Deflection in the plane of greatest resistance began at a load of about 266 kips, the load at which the deflection in the plane of least resistance first began to increase rapidly. From that point, it increased steadily to 0.03 in. at a load of 504 kips, at which time a rapid increase began. At the maximum load of 527 kips, this deflection was 0.06 in. Although the test was continued beyond the maximum load, the instruments had been removed and the deflection was not measured. The principal final deflection took place in the plane of least resistance when, with the testing machine still running, the load had fallen off to about 500 kips. That in the plane of greatest resistance was still not apparent to the eye.

Inspection of Fig. 18 shows that, except at the maximum load, the deflections of the riveted Columns Nos. 2 and 5 were smaller than those of the corresponding Columns Nos. 1 and 4, welded with continuous beads. Fig. 2 indicates that the riveted columns were initially straighter than the continuously welded columns. Fig. 18 shows that Columns Nos. 6 and 7, with intermittent beads, deflected less under load than the corresponding Columns Nos. 3 and 4, with continuous beads. Columns Nos. 6 and 7 were initially slightly straighter than Columns Nos. 3 and 4. Still another comparison may be made. Columns Nos. 4, 5, and 7, were alike except that Column No. 5 was riveted, Column No. 7 was welded with an intermittent bead, and Column No. 4, with a continuous bead. The magnitudes of the initial departures from a straight line increased in the order given—namely, Columns Nos. 5, 7, and 4—whereas the deflection under load was least for Column No. 7 (intermittent weld), next larger for Column No. 5 (riveted), and largest for Column No. 4 (continuous weld).

In all comparable cases the deflections were largest for the columns welded with the continuous bead. Only one comparison between a riveted column and a column with intermittent bead could be made, and the deflection was less for the intermittently welded, than for the riveted, column. The two comparisons available showed the deflection for the intermittently welded, to be less than that for the continuously welded, column.

The columns fabricated by riveting were straighter than those fabricated by either continuous or intermittent welding. The magnitude of deflection under load corresponded to the magnitude of the initial departures from straightness, except that the riveted column (No. 5) gave greater deflection under load than the less straight column (No. 7), which was welded with intermittent beads.

In general, the differences in deflection were small and only one column of a kind was tested. Therefore, it should not be concluded that these statements represent in general the relation between deflections of riveted and welded columns.

8.—Bending Stresses

The averages of the measured strains for Gauge Lines 4 and 12, and for Gauge Lines 16 and 24, were computed separately. The former represents the strain on the west, and the latter, the strain on the east, edges of the cover-plates. One-half the sum of these average strains represents the average strain over the entire cross-section and one-half their difference represents the bending strain in the "extreme fiber," provided that (1) the strains were measured correctly; and (2) a plane section before bending remained plane after bending. These strains, multiplied by 29 000 000 lb. per sq. in. (the average modulus of elasticity for all columns), give the average stress, $\frac{P}{A}$, and the bending stress, respectively.

A comparison of the average stress thus computed, with the total loads divided by the area of the columns—that is, $\frac{P}{A}$—is given in Fig. 19. The two curves for average stress agree closely for the lower loads. This agreement should be expected up to a point at which the maximum stress is near the yield point of the material, but a marked divergence takes place at a value of $\frac{P}{A} = 21$ kips per sq. in. for Column No. 1*BB*, and at a value of 16 kips per sq. in. for Column No. 2 (total loads, 510 and 336 kips, respectively), whereas the yield point of the coupons occurred at a stress of about 35 kips per sq. in. for each of these columns (Table 6). It is possible that, due to the presence of initial stresses, the limit of proportionality between stress and strain had been reached at certain places, under comparatively low average stresses.

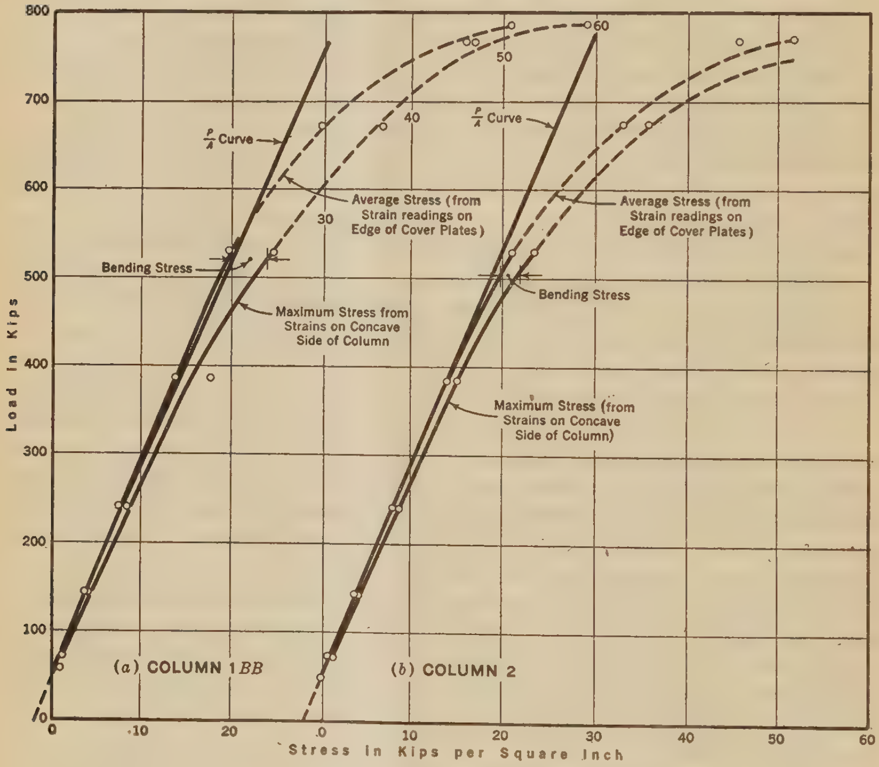

FIG. 19.—LOAD STRESS CURVES.

TABLE 6.—RESULTS OF COUPON TESTS OF COLUMN MATERIAL

Column No.	Sectional Area, in Square Inches			Yield-Point Stresses, in Kips per Square Inch			Stresses at Maximum Load, in Kips per Square Inch		
	I-section	H-section	Plates	Core, section, I or H	Plates	Weighted average	Core section, I or H	Plates	Weighted average
1	......	9.3	15.0	38.8†	32.5†	34.9	62.2†	58.7†	60.1
1*BB*	......	9.3	15.0	38.8†	33.8†	35.4	60.2†	58.6†	59.2
2	......	9.3	15.0	40.8†	32.8†	35.9	61.7†	57.5†	59.1
3	10.22	....	13.75	35.3*	34.1†	34.6	56.8*	58.6†	57.7
4	7.38	....	6.88	38.9*	38.5†	38.7	59.2*	59.4†	59.3
5	7.38	....	6.88	40.3*	38.1‡	39.2	59.8‡	59.8‡	59.8
6	10.22	....	13.75	40.5*	36.0†	37.9	66.4*	65.0†	65.6
7	7.38	....	6.88	39.2*	40.4†	39.8	62.1*	57.2†	59.7
Average.....	......	....	......	39.0	35.8	37.1	61.0	59.3	60.1

* Average of six test coupons. † Average of eight test coupons. ‡ Average of twelve test coupons.

Fig. 8 is a summary of the bending stresses for Columns Nos. 1 to 5, inclusive. The relation between bending stresses at the top, middle, and bottom for Column No. 1 is shown in some detail in Fig. 20. In all cases

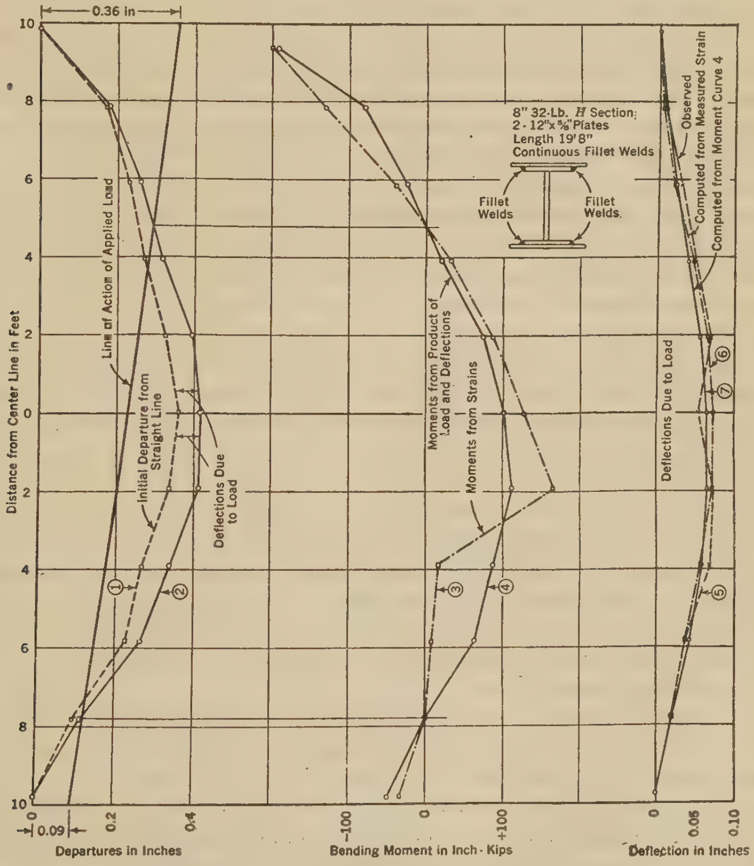

FIG. 20.—DEFLECTIONS AND BENDING MOMENTS FOR A LOAD OF 528 KIPS, COLUMN NO. 1.

the bending stresses at the top and bottom, Sections *BB* and *DD*, are opposite in sign from those on the same side of the column at the mid-section, *CC*, although all were plotted in the same direction for convenience. In Columns Nos. 3, 4, and 5, the bending stresses at the top, middle, and bottom were about equal, as would be the case if the columns were fixed at the top and bottom. In Columns Nos. 1, 1*BB*, and 2, the bending stresses were greater at the top than at the middle, indicating some initial eccentricity of loading in the columns. There is nothing in Fig. 8 to indicate any marked advantage of either riveted or welded columns over the other, as far as freedom from bending moments is concerned.

Strangely enough the bending stress seems to have had little effect on the maximum loads carried. This is shown by the fact that the ratio of the stress at maximum load, to the yield-point stress of the material (given in Table 3), was nearly constant for all columns.

9.—Bending Moments and Deflections in Column No. 1

In the test on Column No. 1, longitudinal strains were measured on the four corners; that is, on the edges of the cover-plates, at sections approximately 2 ft. apart, throughout the length of the column. At the same sections deflections were measured parallel to, and perpendicular to, the web of the column. The deflections due to a load of 528 kips were added to the initial departures from a straight line and the resulting total departures (Fig. 20, Curve 2), indicate the total curvature in the same plane under this load.

It was desired to ascertain whether the deflections of this column under load were those that should be expected with a column having as much initial curvature as this one possessed. This question appears to be of the greater significance as Column No. 1 carried a maximum load (785 kips) slightly in excess of that (784 kips) carried by the much straighter column, No. 1*BB*, which otherwise was similar to it.

For the load of 528 kips, the bending moments about a gravity axis which was parallel to the web of the columns were computed from the measured strains for all sections at which strain-gauge measurements were taken. For these computations the usual formula, $M = \frac{SI}{c}$, was used, in which, M is the moment of the internal stresses; S is the bending stress; I is the moment of inertia of the column section about the gravity axis to which the moment is referred; and c is the distance from the gravity axis to the extreme fiber, that is, one-half the width of the plate. The strain used in computing the stress, S, was one-half the average difference of the strains measured on the opposite edges of the cover-plates. A modulus of elasticity of 29 000 000 lb. per sq. in. was used.

The bending moments computed in this manner have been plotted in Curve 3 of Fig. 20. It will be noted that there were two points of zero moment within the length of the column. Since the bending moment is equal to the product of the load times the distance of its line of action from the center of gravity of the section of the column, the line of action of the load must pass through the center of gravity of the sections of zero bending moment. This fact was used to locate the line of action of the load. Projecting the two points of zero moment horizontally from Curve 3, to intersect Curve 2, two points on the line of action were established. A line passed through these points indicates that there was an eccentricity of the load equal to 0.36 in. at the top of the column and to 0.09 in. at the bottom. Multiplying the load by the departures of its line of action from the center line of the deflected column gives the moments at the various sections. These moments have been plotted in Curve 4 for comparison with those found from the strains.

While there are certain marked disagreements, the general agreement between these two moment curves, 3 and 4, is good. From each moment curve (that is, that obtained directly from the strains, and that from the product of the load times the departures), the deflections have been determined by a double integration of the $\frac{M}{EI}$-curve for the column. The measured deflections and the deflections computed by the two methods have been plotted in Curves 5, 6, and 7, for comparison with each other. Since the deflection curve could have been determined directly from the measured strains instead of first using the strains to compute the moments, the deflection curve, 6, has been designated as that computed from the measured strain. In determining the deflections given in Curve 7 the moments computed as loads times departures were used. The agreement between the two methods (which is very good), indicates that there was nothing mysterious in the behavior of the column so far as deflections are concerned. It is not clear, however, why Column No. 1 carried a slightly larger load than Column No. 1*BB,* in spite of the lack of alignment of Column No. 1. The explanation is not to be found in a difference of the yield points of the steel for the two columns, since this was only 0.5 kip per sq. in. The weighted average yield point for Column No. 1 was 34.9 kips per sq. in., and that for Column No. 1*BB* was 35.4 kips per sq. in. In spite of its greater initial straightness, the maximum bending stress at the load of 528 kips was greater in Column No. 1*BB* than in Column No. 1, and this may account for the fact that Column No. 1 carried slightly more load than Column No. 1*BB*. Possibly, the lack of planeness of the ends of the columns had more to do with the development of bending stresses than the lack of initial straightness.

It was unexpected to find an eccentricity of 0.36 in. for the resultant load at the top and an eccentricity so much greater there than at the bottom. As indicated in the Appendix, Article 2, much care was used in centering the columns. However, as shown in Fig. 8, bending strains were found early in the tests. Although the column ends were milled, they were not plane enough to provide uniform contact with the bearing-plate, and this lack of planeness seems to be the most probable cause of the eccentricity.

10.—Proportional Limit and Maximum Load

The stress at which any curve in Fig. 10 departed appreciably from a straight line was taken as the proportional limit for the column represented by the curve. The values thus determined are given in Table 3. The proportional limits for Columns Nos. 3, 6, and 7, are so low as to raise some doubt as to the validity of the values selected, but for Column No. 3 the points determined by the strain gauge and those determined by the compressometer readings for the full length of the column coincide throughout the curve, giving confirmation to the value chosen as the proportional limit. For Columns Nos. 1*BB*, 2, 4, and 5, the points determined by the strain gauge coincide with those from the compressometer up to a high load. For Columns Nos. 1, 6, and 7, strain-gauge readings for plotting the average strains were not taken, therefore, no comparison between proportional limit by the two methods was available.

It has been indicated quite clearly that the presence of initial compressive stresses at any position hastened the appearance of strain lines at that position. The stresses at the proportional limit, as shown in Table 3, were higher than the stresses at the first appearance of strain lines, yet there is a general correspondence between the two sets of values which suggests that the presence of initial stresses affected the limit of proportionality between applied stress and resulting strain.

Table 3 gives a comparison of the strengths computed by the use of the Rankine formula,

$$\frac{P}{A} = \frac{S}{1 + \frac{1}{25\,000}\frac{l^2}{r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

with the stresses at maximum load. In this formula: P = maximum load; A = sectional area; S = yield-point stress from the coupon tests; $\frac{l}{r}$ = slenderness ratio; and, $\frac{1}{25\,000}$ = an empirical constant for steel columns with fixed ends. The Rankine formula was used for comparison, because it is well known and widely accepted.

The strengths of different columns cannot be properly compared merely on the basis of the stresses developed at the maximum load. It has been found[7] that for columns of the same slenderness ratio, $\frac{l}{r}$, the stresses at maximum load were approximately proportional to the yield-point stress of the steel from which the columns were made. The ratio of the average stress at maximum load on the column to the yield-point stress of the steel thus becomes a measure of the effectiveness with which the material in the column is used. The effectiveness ratio, that is, $\frac{\text{yield point}}{\text{column strength}}$, has been computed and is given in the next to the last column of Table 3. The highest ratio given is 0.954; the lowest is 0.879; and the average is 0.913. The maximum variation from the average is 4.4 per cent. The average effectiveness ratios for the riveted, continuously welded, and stitch-welded columns were 0.909, 0.921, and 0.900, respectively.

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[7] *Technologic Paper No. 328,* Bureau of Standards, p. 85.

On the whole, it must be concluded that the maximum loads stated in terms of the yield-point stress of the material, determined at speed of testing stated, that is, the effectiveness ratios, were so nearly equal regardless of the method of fabrication of the columns, that no reliable distinction can be made between the merits of the three methods represented.

The ratio of the strength, $\frac{P}{A}$, computed by the Rankine formula, to the average stress at maximum load, is given in the last column of Table 3. The values of this ratio range from 84 to 92%, and the average is 88% of the maximum load.

11.—SUMMARY OF RESULTS

The results of this investigation may be summarized in the following eleven items:

(1) The largest slipping of plates shown by the measurements, was about 0.001 in. Even this may have been merely a difference in compressive deformation within the gauge length over which the measurements were taken. No weaknesses attributable to slipping of plates developed.

(2) Stitch welding caused shortening of the metal at sections through the welds and elongation at the edges of the cover-plates at sections midway between welds. In the stitch-welded columns, the strain lines appeared first on the sections through the welds, but none appeared in the welds or in the metal very close to the welds. In general, they did not appear between the welds except in the webs of the columns, where their formation apparently was brought about by initial compressive stresses set up in the cooling of the section after rolling.

(3) The points of first appearance of strain lines corresponded in position to the points of highest stress due to heating of the metal, as far as the stress due to heating could be determined.

(4) The average modulus of elasticity determined from the coupon tests was 29 300 000 lb. per sq. in., and that determined in the tests of the columns was 28 650 00 lb. per sq. in. The modulus used in computing stress from strain was 29 000 000 lb. per sq. in.

(5) In all comparable cases, the deflections were greatest for the continuously welded columns. There was little distinction as to deflection between the riveted and the stitch-welded columns. In general, the magnitude of the deflection corresponded to the magnitude of the initial departures from straightness. However, only one column of a kind was tested, and the differences in deflection are not large. Therefore, conclusions as to deflection should be used with caution.

(6) In Columns Nos. 3, 4, and 5, the bending stresses, and, therefore, the bending moments, were about equal at top, middle, and bottom. In the other columns, the position of maximum moment varied between top, middle, and bottom. Nothing in the tests indicated any marked advantage of either riveted or welded columns over the other as far as freedom from bending

moments is concerned, although the continuously welded columns seemed to be more subject than the others to initial deflection.

(7) In spite of its large initial deflection, Column No. 1, which was continuously welded, carried slightly greater maximum load than its counterpart, Column No. 1*BB*. The bending moments in this column, computed as the product of the loads times the deflections relative to the line of action, agreed fairly well with the moments computed from the measured strains. The deflections computed from the bending moments also agreed very well with the measured deflections.

(8) The limit of proportionality between applied stress and resulting strain occurred at stresses well below the yield point, but higher than the stresses at which strain lines first appeared. There was, however, a general correspondence between the stress at the limit of proportionality and that at which strain lines first appeared, and this suggests that both were influenced by the presence of initial stresses in the columns.

(9) The ratios of the stress at maximum load to the yield-point stress determined from the coupons were so nearly equal for all the columns tested, that no reliable distinction can be made between the merits of riveting, or welding, columns, either with continuous or an intermittent weld.

(10) Although the tests indicated the presence of initial stresses of considerable magnitude, introduced in the cooling after welding (and probably after rolling also), the maximum loads carried did not appear to be appreciably influenced thereby.

(11) The average stress at maximum load was about 15% greater than the strength computed by the Rankine formula, Equation (1).

ACKNOWLEDGMENT

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APPENDIX

1.—Test Coupons.—A piece 1 ft. long was cut from each rolled section (plates, I-beams, and H-beams) before the columns were shipped from the fabricating shop. From this piece, four test coupons were taken for each cover-plate, six for each I-section, and eight for each H-section. These coupons were tested in tension for yield point and ultimate strength, and a few for modulus of elasticity. Hence, the yield point and ultimate strength of the steel reported for each column is based on the average of fourteen or sixteen coupons. As a result of this procedure the columns were 1 ft. shorter than the lengths originally designated.

Fig. 21 shows by cross-hatching the location and numbering of the test coupons cut from the various sections. The letter, *A*, when present, indicates that the coupon was taken from the core, that is, from the I-section or the H-section. The letter, *P*, indicates that it was taken from one of the plates. The final numeral indicates the location in the section. The coupons were arranged with a view of determining whether there were systematic variations of yield point throughout the section. All specimens were 1½ in. wide, except *A*1, *A*3, *A*5, and *A*7; and Columns Nos. 1, 1*BB*, and 2, which were 1¼ in. wide.

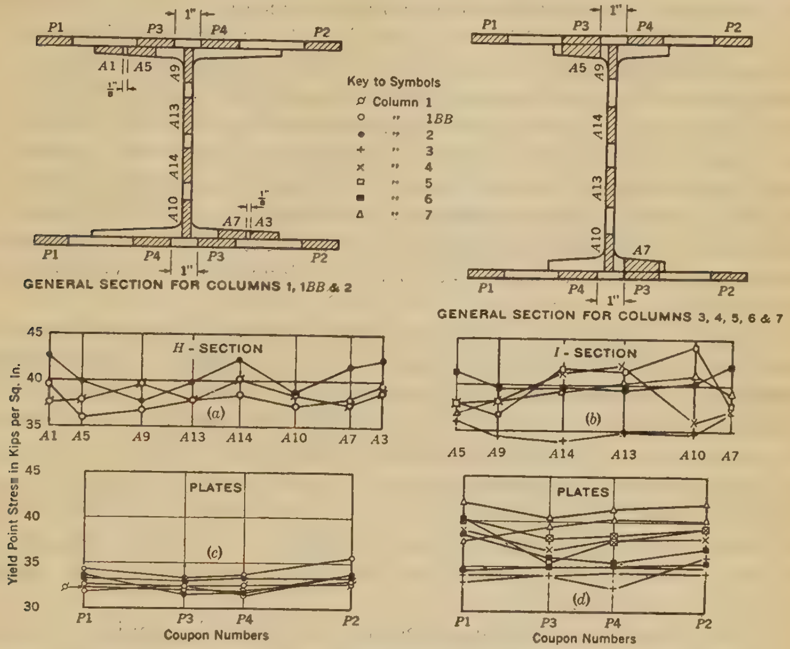

FIG. 21.—YIELD-POINT STRESSES DETERMINED FOR COUPONS.

The tests of the coupons were made in a 50 000-lb. testing machine in the usual manner for tension tests. The yield-point load was determined by the "drop" of the weighing beam in the tests in which the modulus of elasticity was not determined. In the modulus tests, the yield point was chosen as the load for which the strain indicated by the gauges showed a rapid increase with little increase in load. An extensometer with a gauge length of 2 in. was used in the tests for modulus of elasticity.

Considerable attention has been paid to the determination and weighting of yield-point stresses, because the strength of the column has been found to be more directly a function of the yield point than of the ultimate strength of the materials from which the column was made.

The yield-point stresses for individual coupons have been plotted in Fig. 21. The horizontal distances between points are proportional to the distances from center to center of coupons, so that a horizontal line on the diagram representing the average yield point for all the coupons for a given plate, will be weighted approximately according to the sectional areas represented by the several coupons. Fig. 21 (*a*) shows a tendency for the higher yield points to occur at the edges of the flanges and at the center of the web, and a lower yield point at the roots and at intermediate points on the flanges. However, examination of Fig. 21 (*a*) to Fig. 21 (*d*) indicates that the variation in yield-point stress for different locations in any section (either plate or core section) from the average, was not greater than about 2 kips per sq. in. An average not weighted for the area represented by individual coupons was used, therefore, as the yield-point stress for each section. The average yield points for the plates, however, differed in general from those for the corresponding cores, and it was necessary, in determining the yield-point stress for a column, to weight the yield point of the plates and that of the core in proportion to the area of each. This has been done, and the results are given in Table 6.

The yield points for the three **H**-sections show little variation from each other, while the yield points for the **I**-sections showed a somewhat greater variation, due to the low yield point for the **I**-section in Column No. 3. Among the plates the variation of yield point was greatest of all. Columns Nos. 6 and 7 were furnished considerably later than the other columns, and the yield points of the plates in these columns were considerably greater than the average for all plates. With only one exception, the yield point of the plates was lower than that for the core section in the same column. The maximum variation of an individual from the average of all the weighted yield points was about 7 per cent.

2.—Testing.—The following description of methods applies to Column No. 4 which was the first column tested. The others were tested in the same manner except for minor changes and improvements, as follows:

(*a*) Two-inch strain-gauge readings (Section *AA*, Fig. 6) were omitted from the other column tests.
(*b*) Four more gauge lines at Section *DD* in which slipping of the plates was measured, were added in the later tests.
(*c*) Gauge Lines 1, 3, 13, and 15 were added at Sections *BB* and *DD*.
(*d*) The testing of Column No. 1 differed materially from that of the other columns. This test is described in Section 9 of the main body of this paper.

The locations of the gauge lines on the cross-section of Column No. 4 are shown in Fig. 6. The gauge length at Section *AA* was 2 in.; at Sections *BB*, *CC*, and *DD*, it was 10 in. The gauge holes were drilled with a No. 54 drill (0.054 in. in diameter). Gauge Lines 5, 11, 17, and 23 in Section *BB*, were on the edge of the **I**-beam which formed the core of the column. Gauge Lines 5*b*, 11*b*, 17*b*, and 23*b*, had the same upper holes as Gauge Lines 5, 11, 17, and 23, but the lower hole for each of these lines, instead of being in the **I**-beam, was in the cover-plate close to the fillet weld. Equality of strain in

Gauge Line 5 with that in Gauge Line *5b*, would indicate that there was no slipping of the plate, but any difference in the strains between Gauge Lines 5 and *5b* indicates slipping of the plate relative to the I-beam, and this is true also with the other similarly placed gauge lines.

Before placing Column No. 4 in the testing machine, it was whitewashed with a mixture of white Portland cement and water, in order to make more evident the flaking of the mill scale (herein termed "strain lines") as the column came under stress.

After it had been placed in the machine, and before any load had been applied, provision was made for measuring the shortening over the entire length of the column at its four corners, and for measuring the center deflection relative to the ends. The instruments for measuring this shortening are here termed "compressometers" to distinguish them from the strain gauge. Each compressometer consisted of an Ames gauge and a distance piece. The distance piece was a $\frac{3}{8}$-in. steel pipe attached to the column 1 in. below the top, and of such length that its lower end bore against the plunger of the Ames gauge fastened 1 in. above the bottom of the column. Thus, the compressometers measured the total shortening of a column under load, except that which occurred within the 2 in. outside the gauge lenth. The distance piece was held in line by guides at about the one-third points of the length of the column. The Ames gauges read satisfactorily to thousandths of an inch. Each 0.001-in. total shortening corresponds to an average strain of $\frac{0.001}{182}$, or 0.0000055 in. per in. for the 182-in. gauge length, or to an average stress of 160 lb. per sq. in., using a modulus of elasticity of 29 000 000 lb. per sq. in.

For measuring the deflections, two fine wires were attached to the web of the column 6 in. below the top and 6 in. above the bottom, giving a span of 172 in. The deflection was read to the nearest 0.01 in. on scales attached to the column. The arrangement is shown in Fig. 22.

The column was tested in a standard two-screw testing machine of 800 kips capacity. As placed in the machine, the web of the column was in the plane of the longitudinal center lines of the screws. The bottom of the column was placed on a planed steel plate 3 in. thick, which rested directly on the bed of the machine.

This column was placed so that its top was centered accurately between the screws, letting the bottom of the column come where it would. Mounted on top of the column was a 3-in. plate, planed on both sides. On top of the plate and directly under the head of the compression block, were three pairs of wedges, each resting on a spherical bearing block. The three spherical bearing blocks, as placed on the 3-in. plates, outline an isosceles triangle, the apex block being centered over the web, 4$\frac{3}{8}$ in. north of the center line of the column. The other two bearing blocks were 4$\frac{5}{8}$ in. south of the center line of the column, 8 in. apart, center to center.

The compression head of the testing machine was then brought down until the wedges were nearly in contact with it; the wedges were driven in until they were equally tight as nearly as this could be judged; a load of 15 kips was applied to the column; and the compressometer readings were taken on all four edges of the column. This was followed by a load of 60 kips after which the compressometers were again read. If the difference in shortening on the four edges of the column revealed an eccentric loading, the compression head was raised, and the wedges were re-adjusted. This procedure was repeated until the shortenings measured by the four compressometers were equal within 0.002 in. in the 182-in. gauge length. The loading was then considered to be nearly enough axial to permit proceeding with the test.

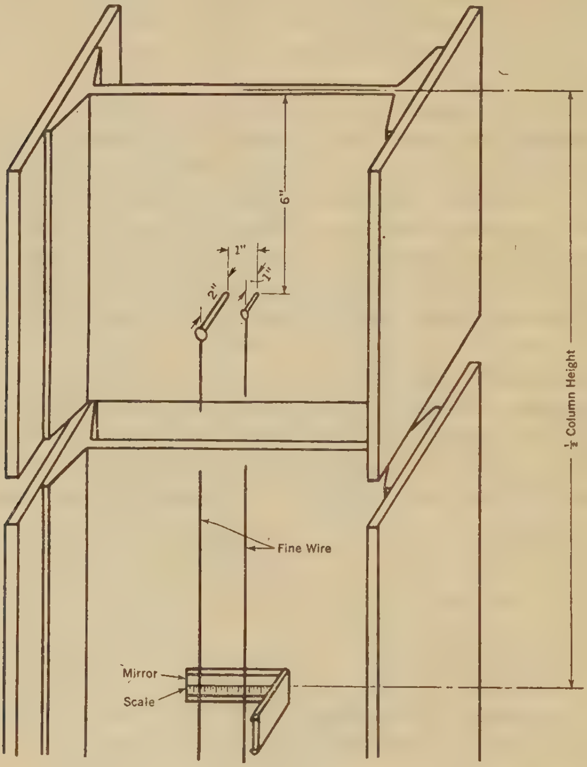

FIG. 22.—DEFLECTION APPARATUS.

The test was carried out with the increments of load shown in Fig. 18. Readings were taken on the compressometers and the deflection apparatus at more frequent intervals than on the strain gauges.

In the case of Column No. 4, after reaching the maximum load the operation of the testing machine was continued until the load had decreased to about 500 kips. Column No. 8 was not loaded to its maximum. In all other cases the loading was discontinued as soon as it was evident that the maximum load had been reached.

DISCUSSION

Raymond J. Roark,[8] Esq. (by letter).—The part of this paper that deals with the effect of welding on strains and stresses is of special interest to the writer, and it is with this subject that this discussion is concerned.

That the unequal heating caused by welding the parts of a composite member produces residual stresses is obvious, and many have expressed apprehension as to the possible effect of such stresses on the strength and stiffness of members so fabricated. Tests have been made for the purpose of securing information on this point, but few of them seem to have been planned so as to yield information on all three of the questions of practical importance, namely: (1) What total strains are produced by a given welding procedure? (2) What total stresses are produced? (3) What are the effects of these strains and stresses on the mechanical properties of the member as a whole? Thus, the tests made by Messrs. Slater and Fuller revealed the magnitude of the total strains produced and the effect—or absence of effect—on the mechanical properties of the columns; but, as the authors point out, they did not show what part of the total strain was plastic and what part represented elastic deformation accompanied by residual stress.

Some time ago the writer made tests designed to throw light on all three of the foregoing questions as they applied to the so-called "button" or rivet welding. The connection, in this type of weld, is effected by punching or drilling holes in one of the parts, placing it against another part, and filling or partly filling the holes with weld metal, as shown in Fig. 23. It is a convenient substitute for riveting, and is especially suitable for building up composite beams or colums of plates and channels.

The specimens used are shown in Fig. 24. The scheduled procedure in making up the specimens and the corresponding standard test pieces was as follows:

(1) Make six specimens from three bars; namely, Specimens *A* and *B* from one bar, Specimens *C* and *D* from one bar, and Specimens *E* and *F* from one bar.

(2) Make one standard tensile specimen from each bar.

(3) Anneal all material, including tensile specimens, after cutting.

(4) Drill Specimens *A*, *C*, and *E* for strain-gauge holes before welding and take strain-gauge readings before and after welding.

(5) Weld Specimens *A*, *B*, *C*, and *D*, thus:

(*a*) Weld Holes 1 and 5 (Fig. 24) and allow 30 min. for cooling.

(*b*) Weld Holes 3 and 6, and allow 30 min. for cooling.

(*c*) Weld Holes 2 and 4.

(6) Weld Specimens *E* and *F* without cooling and, in the sequence, Holes 1, 5, 3, 6, 2, and 4.

(7) Saw Specimens *A*, *C*, and *E*, as shown in Table 7, and then take strain-gauge readings.

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[8] Associate Prof. of Mechanics, Univ. of Wisconsin, Madison, Wis.

(8) Make tensile tests with extensometer on Specimens *B, D,* and *F,* and on the standard tensile specimens.

(9) Make two additional specimens, *G* and *H*; follow the welding procedure outlined in Steps 2, 3, 4, 5, and 7, but anneal after welding and then test in tension.

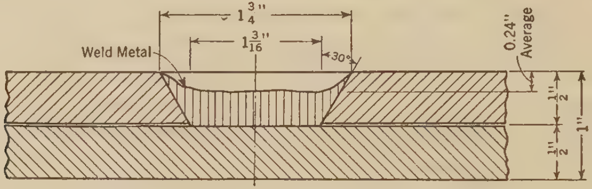

Weld Metal
$1\frac{3}{4}$"
$1\frac{3}{16}$"
30°
0.24" Average
$\frac{1}{2}$"
$\frac{1}{2}$"
1"

FIG. 23.—SECTION *A-A,* IN FIG. 24.

Referring to Step 7, strain-gauge readings were taken on Specimens *A, C,* and *E* before and after welding, measurements being made on the gauge lines indicated in Fig. 24, and the total strain consequent upon welding was thus determined. The plates were sawed through as directed by Step 7 to ascertain how much of this strain was elastic. The front plates (with holes) were cut on the solid lines and the back plates (without holes) were cut on the dotted lines. A third set of strain-gauge readings was then taken on the uncut plate, and the elastic recovery thus revealed made possible an approximate calculation of the actual residual stresses introduced into the plates by the welding operation. Of course, only average stresses over the 2-in. gauge lengths could be determined thus, and it was possible to ascertain neither the maximum stresses nor the manner in which the stress varied from point to point.

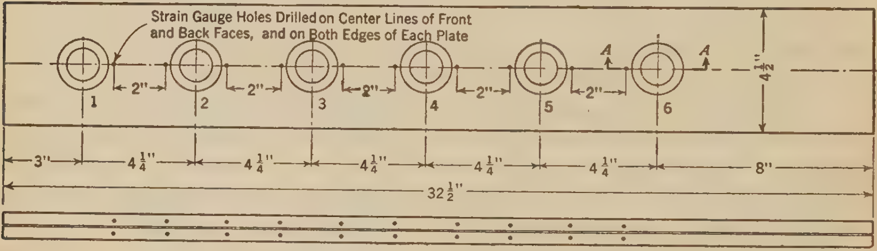
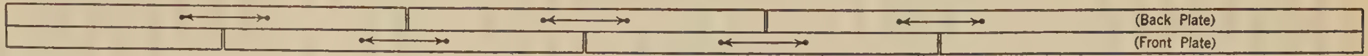

Strain Gauge Holes Drilled on Center Lines of Front and Back Faces, and on Both Edges of Each Plate
A
A
2"
2"
2"
2"
2"
1
2
3
4
5
6
$4\frac{1}{2}$"
3"
$4\frac{1}{4}$"
$4\frac{1}{4}$"
$4\frac{1}{4}$"
$4\frac{1}{4}$"
$4\frac{1}{4}$"
8"
$32\frac{1}{2}$"

FIG. 24.—TYPICAL TEST SPECIMEN.

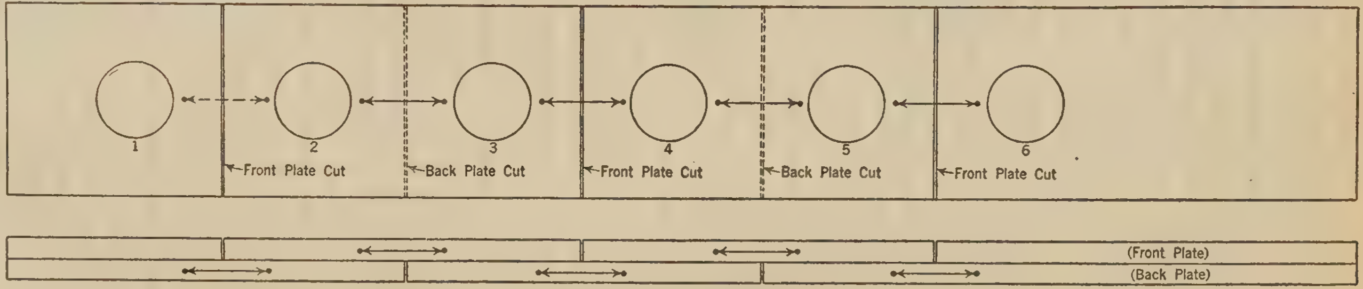
The results of these measurements and calculations are given in Table 7. For convenience, the product of unit deformation and modulus of elasticity is given, rather than the unit strain. When this product is calculated on the basis of total strain (plastic and elastic), it is called apparent stress and when calculated on the basis of elastic strain, it is called actual stress. The terms are not exact, but it is believed that their significance is clear. In Table 7 these apparent and actual stresses are tabulated adjacent to the gauge lines where they were found to occur.

TABLE 7.—STRESSES MEASURED IN TEST SPECIMENS, IN THOUSANDS OF POUNDS PER SQUARE INCH

Specimen	Back plate	Front plate	Back plate	Front plate	Back plate
			APPARENT STRESS; UPPER EDGE		
A	—17.2	—12.0	—15.0	—14.3	—12.7
C	—13.5	—12.0	—12.7	—11.2	—14.3
E	+14.2	+ 6.0	—39.7	—18.0	—76.5
			ACTUAL STRESS; UPPER EDGE		
A	— 2.25	+ 2.25	0	— 0.75	— 6.75
C	— 1.5	+12.0	— 4.5	+ 8.25	0
E	+10.5	— 7.75	—10.5	— 1.5	+22.5


(Back Plate)
(Front Plate)

Specimen	Back plate	Front plate	Back plate	Front plate	Back plate
			APPARENT STRESS; FACE		
A	+67.5	+84.0	+65.0	+76.5	+66.0
C	+49.5	+77.2	+70.0	+64.0	+60.0
E	+40.5	+82.5	+45.75	+73.0	+93.0
			ACTUAL STRESS; FACE		
A	+ 3.0	— 8.25	+ 5.25	0	+ 6.0
C	0	+ 5.25	+12.2	+10.5	+ 1.5
E	0	0	0	— 2.25	+ 6.0


1
2
Front Plate Cut
3
Back Plate Cut
4
Front Plate Cut
5
Back Plate Cut
6
Front Plate Cut
(Front Plate)
(Back Plate)

Specimen	Back plate	Front plate	Back plate	Front plate	Back plate
			APPARENT STRESS; LOWER EDGE		
A	—30.0	—9.75	—11.2	—11.2	—12.0
C	—10.5	—5.25	— 3.75	—13.5	—12.0
E	—167.5	—9.0	+18.0	0	+42.0
			ACTUAL STRESS; LOWER EDGE		
A	—3.0	+8.25	—0.75	0	—0.75
C	—1.5	+3.0	+3.75	0	—9.75
E	0	+8.25	—4.5	0	+2.25

A study of Table 7 shows that: (1) The apparent stresses were very large; (2) the actual stresses were large in some cases, but were very much less than the apparent stresses, and had little if any consistent relationship thereto; (3) both apparent and actual stresses were highly irregular, showing considerable variation as between different specimens and as between different gauge lines on the same specimen; (4) in general, the actual stress in the intervals between welds was compressive in the rear plates and tensile in the front plates; and (5) as between the standard welding procedure, used in preparing Specimens *A* and *C*, and the "rapid" welding used in preparing Specimens *E*, the standard procedure appears to have caused much smaller apparent stresses and somewhat smaller actual stresses.

Five specimens were tested in tension. Three of these, *B*, *D*, and *E* were, respectively, similar to Specimens *A*, *C*, and *E*. The two others, Specimens *G* and *H*, were similar to Specimens *A* and *C*, but were annealed subsequent to welding. The quantities determined in the tensile test were the modulus of elasticity, proportional limit, apparent elastic limit, and yield point. In addition, the load at which the plaster wash began to flake off, and and the points at which such flaking first occurred, were noted. The capacity of the machine used was not great enough to permit the ultimate strength to be ascertained.

The results of the tensile tests, together with the results of tensile tests on standard coupons of corresponding material, are listed in Table 8, which is arranged to facilitate cross-reference between the actual and apparent initial stresses found in the several types of specimen and their tensile properties. It is apparent from these results, and from the stress-strain diagrams shown in Fig. 25, that the welding operation had little effect upon modulus of elasticity, apparent elastic limit, or yield point, but that it did cause a considerable reduction in the limit of proportionality, in comparison with either the annealed Specimens *G* and *H*, or the coupons.

It appears reasonable to assume that the low elastic limit of the welded test pieces was due to the presence of initial stresses, the material under initial stress reaching its elastic limit at a comparatively low average load. The rather well-defined break in each of the stress-strain diagrams would also seem to indicate that a considerable volume of material reached its elastic limit at the same time, and, therefore, must have been under an approximately uniform intensity of initial stress.

When the results of these tests are compared with those reported by Messrs. Slater and Fuller, it is seen that the apparent stresses are of about the same order of magnitude. The volume of weld metal deposited, and the corresponding heat input, was proportionately greater in the case of the tension specimens, and as would be expected the greatest strains caused were somewhat greater than those found in the columns. With the exception of a few values, however, the difference is not great, and it is of interest to note that the reduction in proportional limit caused by welding was about the same in the columns as in the tension specimens.

It would not be expected that the yield point in tension of a test piece as a whole would be affected appreciably by initial stresses, because the relatively large unit deformation occurring at and near the yield point permits an equalization of stress; but it might be supposed that, in the case of a column, any considerable non-uniformity in stress distribution would lead to buckling at a load less than that which the member might otherwise be expected to carry. The tests reported by Messrs. Slater and Fuller, however,

TABLE 8.—RESULTS OF TENSILE TESTS ON WELDED SPECIMENS COMPARED WITH STANDARD TENSILE SPECIMENS

WELDED SPECIMEN:		MODULUS OF ELASTICITY, IN MILLIONS OF POUNDS PER SQUARE INCH		UNIT STRESSES, IN KIPS PER SQUARE INCH						
Tested in tension	Tested for residual strain	Welded specimen	Standard tensile specimen	Proportional Elastic Limit		Useful Limit Point		Yield Point		Point at which flaking was first observed
				Welded specimen	Standard tensile specimen	Welded specimen	Standard tensile specimen	Welded specimen	Standard tensile specimen	
*B**...	*A**.....	29.0	29.0	13.25	29.6	27.5	30.8	32.45	33.75	32.2§
*D**...	*C**.....	29.25	28.3	14.25	29.6	26.2	30.8	33.0	33.75	36.7‖
F†...	*E*†.....	27.0	29.3	13.25	27.0	28.15	31.4	33.7	34.2	36.4¶
G‡...	*A* and *C*‡	....	....	27.25	....	28.25	....	30.0	....	....
H‡...	*A* and *C*‡	....	....	30.0	....	30.6	....	32.5	....	....

* Welding procedure was normal; that is, time was allowed for natural cooling.
† Rapid welding.
‡ Welding procedure same as for Specimen *B*, except that specimens were annealed after welding.
§ Flaking first observed on the side opposite the welds.
‖ Flaking first observed on the side opposite Welds 4 and 5.
¶ Flaking first observed on the side opposite Welds 2, 6, and 4.

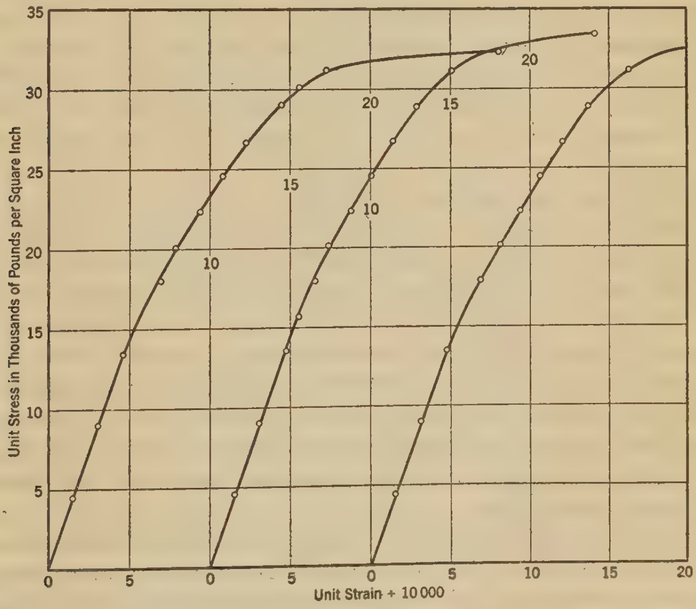


FIG. 25.—STRESS-STRAIN CURVES FOR WELDED SPECIMENS *B*, *F*, AND *D*, TESTED IN TENSION

fail to show any reduction in strength due to initial stress. It is to be noted, in this connection, that the columns tested by the authors were symmetrical with respect to the longitudinal axis, and that the welds also were symmetrically disposed with respect to that axis. This circumstance would tend to minimize both the tendency of the welding stresses to distort the column and their tendency to cause buckling due to local over-stressing. In a column in which neither the material nor the welding was symmetrical with respect to the longitudinal axis, both these tendencies might be appreciable. The welding procedure followed in preparing the tension specimens might have been expected to cause crookedness, and the uneven distribution of initial stresses would appear to indicate that there was some bending; but as far as could be ascertained by the eye the specimens were straight; there was no appreciable curvature either of the complete specimen or of the component plates.

On the whole, both the column tests and the tension tests appear to indicate that while welding may cause large plastic strains and fairly large stresses, and may appreciably lower the proportional limit, these effects are not necessarily accompanied by a reduction in strength. It would be of interest to know whether tests on columns of non-symmetrical section would bear out such a conclusion. It might also be of interest to ascertain the effect of the local over-stressing occasioned by welding on the "damping coefficient" of members thus fabricated.

LYNDON F. KIRKLEY,[9] JUN. AM. SOC. C. E. (by letter).—The commendable and complete analysis of steel column test data in this paper adds materially to the information already available on this subject. It provides needed experimental data on both welded and riveted steel column construction, and it is only to be regretted that the investigation could not have been more extended, especially with regard to the number of test specimens of each type as well as a wider range of core sections.

Although these tests will provide valuable information for a large part of light building construction, until similar tests have been performed on the much larger specimens used in heavy building construction engineers will not know much more of the mysteries of stress action and distribution in built-up steel columns. In heavy column construction it is not uncommon to find the heaviest of rolled **H**-sections buried beneath 2 to 4-in. cover-plates and surrounded by an additional arrangement of rolled angles and plates.

Until improvements have been made in the accuracy of correlation of stress analysis between these more complex column sections and the simpler types, as tested and described in this paper, not much will have been added to heavy steel column data.

Items (2), (3), and (10) under "Summary of Results," emphasize another problem of steel manufacture—the effect of heat upon the parent metal. The authors have pointed out in Item (2), the effects of welding upon the parent metal local to the point of the weld, which were caused by heating and cooling. In addition, the change in metallurgical structure of the parent metal local to the weld may be of some consideration, depending on the thickness of the two welded parts, the size of welding electrode used, arc voltage and amperage, and the rate of deposition of the weld. The effects of these conditions may vary with the thickness of the welded components and the consequent dispensation of heat of welding.

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[9] Eng. Apprentice, McClintic-Marshall Corp., Bethlehem, Pa.

Variation of the chemical composition of the weld metal itself is also possible, depending on the degree of contamination by atmospheric gases during deposition. Mr. J. C. Hodge has pointed out[10] that by varying the efficiency of the protection, either by varying the nature and thickness of surface coatings and coverings, or by changing the length of the arc, the composition of the deposited weld may be changed at will. Thus, the nitrogen content may vary from 0.015 to 0.15, depending on the degree of protection. According to Mr. Hodge, other changes in chemical composition occur during weld deposition; for example, the carbon and manganese content are considerably lowered, which indicates the extreme oxidizing conditions within the arc. Furthermore, variations in chemical composition affect the elastic properties and the behavior under plastic deformation of the various weld metals.

The physical composition of a single ingot from which structural shapes are rolled may also vary from section to section throughout a specimen. Such variations manifest themselves in such natural defects of ingots known as piping, blow-holes, segregation, and crystallization, any of which may be present and vary in size and shape from microscopic to several inches in one dimension. These defects may or may not be corrected during subsequent working of the metal. Consequently, any one or a combination of these influencing factors may account partly for the variation of the yield point among the cover-plate and core-section coupons as mentioned by the authors in Section 1 of the Appendix.

The writer is fully aware that most of the points mentioned herein are of minor consequence with respect to strength of welded and riveted steel columns as finally produced, and that they are usually taken care of by adequate mill and shop inspections; yet he feels that they are deserving of mention and are points that contribute to the explanation of variations noted by the authors.

R. A. CAUGHEY,[11] M. AM. SOC. C. E. (by letter).—It occurs to the writer in reading this paper that it brings out the fact that there is much to be learned regarding the effect of welding in steel. There seems to be abundant evidence that permanent deformation is one of the results of welding. In Conclusion (2) the authors note that stitch-welding caused shortening of the metal at sections through the welds and elongation at the edges of the cover-plates at sections midway between welds. They also show that the moduli of elasticity of the columns as tested were lower than those obtained in coupon tests. It would seem that there may be a definite relation between these two phenomena; namely, that on account of unequal shrinkage in cooling, certain parts of the column sections received an initial compressive stress and other parts some initial tension. It is possible that when load was applied, the compression sections began to take compressive stress immediately, but that the sections in tension did not, until their initial tension had been overcome. Naturally, this would cause more deformation in the column than would exist if the entire section was acting; and, therefore, would give a lower modulus of elasticity for the compound section.

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[10] *Engineering News-Record,* October 13, 1932.

[11] Prof. of Structural Eng., Iowa State Coll., Ames., Iowa.

As is stated by the authors, the two moduli do not differ radically, but the difference seems to indicate that initial stresses due to welding do exist and that they affect the modulus in the manner mentioned. Referring to Table 5, it will be noted that the moduli for the two riveted columns, Nos. 2 and 5, are 28 800 000 and 29 100 000, respectively, while those of welded columns (omitting Column No. 3 which seems to discount to some extent the writer's hypothesis) vary from 26 000 000 to 28 400 000. It should also be noted in connection with this table that the determination of the moduli, in cases of Columns Nos. 1, 6, and 7, is not entirely satisfactory to the authors.

Recently the writer had a little experience with the effect of welding on some small **H**-columns. It was desired to weld some small bars to the columns for the purpose of carrying clinometers to measure rotation of joints. In this case oxy-acetylene welding was utilized, and it is likely that a larger amount of metal was heated than would have been the case in electric welding. It did not take long, however, to find out that all strain-gauge holes near the welding had to be re-drilled, as the gauges would no longer fit into them. The welding process was abandoned in favor of tap-bolts.

If it was possible to do so, it would seem advisable to test some short lengths of compound sections, the specimens to be arranged in such a way that welds could be released without re-heating and stresses measured in various sections separately. It might be possible in this way to get some ideas as to initial stresses in the various shapes of the section. It might also be advisable to make some coupon tests from material adjacent to, or forming, welded connections in order to obtain data on possible change of moduli.

It is possible that changes in moduli may be caused by uneven flow of metal in various parts of a compound section after welding is completed. For instance, perhaps the metal at the junction of the web and flange of the **H** or I-sections of the columns under consideration would be somewhat more resistant to deformation than that in the outer ends of the plates riveted or welded to them. Fig. 16(*a*) seems to indicate that this is true in the tests under consideration.

The writer feels that more should be known regarding moduli changes and initial stresses caused by welding. Testing such sections as were tested by the authors, with provision for releasing welds and making coupon tests of material that has been welded, would probably give some information of that kind.

E. G. WALKER,[12] M. AM. SOC. C. E. (by letter).—The writer has read this paper with considerable interest and hopes that the comparison between riveted and welded structural steel work instituted therein will be continued. The paper evidences the considerable care and attention to detail that has been bestowed upon the work. It shows also that in the study of so uncertain an article as a composite structural steel column it is difficult to draw conclusions except after testing a large number of specimens. The investigation in the present case has been brought to a stage that enables one to judge the main structural differences resulting from the adoption of welding in place of riveting for built-up columns. It is insufficient, however, to enable definite distinctions between the two types to be formulated.

In the tests on the slipping of plates, no instance of undoubted slip of definite magnitude occurs. The fact that all the types tested behaved nearly alike does not enable logical conclusions to be drawn. Unfortunately, no tests are available in which slip actually occurred. Had there been, it might have been possible to deduce conclusions therefrom as to the relative resistance of the riveted and welded columns to slipping. Further experiments are necessary before it can be decided whether, if slip should actually occur in a riveted column, it would also take place in a corresponding welded column.

The use of welding, by reason of the localized heating of the structural material inherent in the process, introduces a new factor which does not have to be considered in connection with riveted work, where, although there must be a limited amount of localized heating around the rivet hole, this heating cannot extend to a sufficient volume of metal to influence stress distribution. The authors' investigations indicate that a certain amount of deflection is caused by stitch-welding and, therefore, there must be a corresponding amount of initial stress in the material. The question arises, as to whether, if heavier and more complicated sections than those used in the experiments were built up by welding, there might not be the possibility of developing initial stresses of such magnitude as to be a source of structural weakness. This does not seem likely, but it is a point that might well be investigated in the future.

The results of the deflection measurements are somewhat remarkable, in that they indicate a greater deflection in the case of the continuous welded column than in the other two cases. The writer would have expected the relative magnitudes to be the other way round, if there was any difference at all. It is quite possible that the limited number of tests available in this case have induced erroneous conclusions. The authors' strain measurements have led them to conclude that there was no definite slip in any of the columns. If that is so, there would not appear to be any reason for variation in the amount of deflection, and it is possible, therefore, that the conclusion may have been arrived at because the number of specimens of which records are available, is insufficient to enable true relative averages to be obtained.

The comparison of actual and calculated maximum loads given in Table 3, is of subsidiary interest. The values of the ratio of these stresses are thoroughly consistent, and the range of variation is not greater than one would expect. It follows, therefore, that the Rankine formula, with the constant used by the authors, gives definitely a strength lower than the actual strength of the specimen. An attempt made to adjust the constant in the denominator of Equation (1) which is the only constant that can be varied (the factor, S, being determined directly from the experimental results), was not altogether successful, and showed that, instead of being $\frac{1}{25\,000}$ as given, the figure should be about $\frac{1}{65\,000}$. This is inconsistent with the generally published values of this constant. It would appear that there is some other factor that should be considered in relation to the test results. Probably the degree of fixity of the ends of the columns when set up for test is the controlling factor.

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[12] (Maxted & Knott), London, England.

M. Holt,[13] Jun. Am. Soc. C. E., and R. G. Sturm,[14] Assoc. M. Am. Soc. C. E. (by letter).—The deflections of columns with known eccentricities have been measured by the writers and have been found to agree closely with those computed by Fidler's formula:[15]

$$D = \frac{Pe}{P' - P} \quad \text{.........................} (2)$$

in which, D is the deflection of the column, in inches; P, the load on the column, in pounds; P', the critical load according to Euler, in pounds; and e, the total initial eccentricity, in inches.

A study of Equation (2) shows that the load-deflection curve for a column is curved from the first, thereby making the selection of a proportional limit from the load-deflection curve very indefinite. This probably contributed to the difficulty in selecting the proportional limits for the columns tested by the authors.

All columns have some eccentricity, which may be accidental and unintentional and may be made up of the sum of the effects of crookedness, non-homogeneity of material, uneven bearing at the ends, etc. The initial eccentricity will be large or small depending on whether the foregoing effects are all in the same direction or whether they counteract one another. Some of the columns reported in this paper were apparently quite straight while Specimen No. 4, for example, appears to have been quite crooked. The total initial eccentricity can be found from load-deflection data by plotting $\frac{1}{D}$ against $\frac{1}{P}$ and interpreting the data in the light of Fidler's formula which may be re-written as:

$$\frac{1}{D} = \frac{1}{P} \times \frac{P'}{e} - \frac{1}{e} \quad \text{.........................} (3)$$

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[13] Research Engr., Aluminum Research Laboratories, Arnold, Pa.

[14] Research Structural Engr., Aluminum Research Laboratories, New Kensington, Pa.

[15] Fidler's "Treatise on Bridge Construction."

Equation (3) was first used in this form by Professors W. E. Ayrton and John Perry.[16] It may be noted that the intercept of this straight line on the $\frac{1}{D}$-axis is $\frac{-1}{e}$, and the intercept on the $\frac{1}{P}$-axis is $\frac{1}{P'}$. Information picked from the plotted data given in Fig. 18 of the paper is not sufficiently accurate to yield a reliable determination of the initial eccentricities of the specimens.

The fact that the bending stresses reported in the paper seem to have little effect on the maximum loads carried by mild steel columns is referred to as strange. It may be noted that in the cases shown, the bending stresses were relatively small and that if they are added to the ultimate strengths of the columns the total stresses would be quite close to the yield point stresses. Because of the definite yield point of the steel in the columns, the maximum stresses would not have an opportunity to increase to values appreciably above the yield strength of the material without excessive bending deformations. Thus, the loads giving maximum computed stresses equal to the yield strengths agree closely with the values obtained from the tests.

M. O. FULLER,[17] M. AM. SOC. C. E. (by letter).—The discussion of this paper has been a tribute to the memory of the late Willis A. Slater, M. Am. Soc. C. E., who was interested in the Society for many years, had contributed papers to its publications, and had taken an active part in discussion.

The tests described by Professor Roark are interesting in that they throw light on the total strains and stresses produced by a given welding process and their effect on the mechanical properties of the member as a whole. The fact that the apparent stresses from the tensile and compressive tests are about the same order of magnitude is very interesting as it may be of service to the designer in indicating the effect of the two kinds of stress upon welded connections.

The points mentioned by Mr. Kirkley are, as he states, of minor importance with respect to the strength of the welded and riveted steel columns, and are usually taken care of by careful inspecton.

Considerable is still to be learned regarding the effect of welding, and investigators are carrying on tests in this field. The work reported in the paper was only a small contribution to this subject, and the writer agrees heartily with Professor Caughey that much needs to be done to establish the relation between the initial tensile and compressive stresses set up on account of unequal shrinkage in cooling.

The question of crookedness in the test columns was one that had not been anticipated and for that reason the original column that was crooked was replaced by another made at a later date. The original column (No. 4) was tested simply as a matter of routine, as it was realized that one test was not sufficient to draw conclusions to any extent. Messrs. Holt and Sturm have explained the reason for the difficulty in selecting the proportional limit of the columns. A full report on tests of columns with known eccentricities referred to by them would be valuable.

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[16] "On Struts," *The Engineer* (London), December 10 and 24, 1886.

[17] Associate Prof., Civ. Eng., Lehigh Univ., Fritz Eng. Laboratory, Bethlehem, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1860

THE MARTINEZ-BENICIA BRIDGE[1]

BY W. H. KIRKBRIDE,[2] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. N. F. HELMERS, E. J. SCHNEIDER, M. F. CLEMENTS, JACOB FELD, AND W. H. KIRKBRIDE

SYNOPSIS

This paper contains a description of a double-track railroad bridge across Suisun Bay, California, constructed by the Southern Pacific Company. It replaces train-ferry operations across the Carquinez Strait between Benicia and Port Costa, Calif. The project includes other steel structures, in addition to the main bridge, and about 20 miles of railroad track equipped with signal and interlocking plants, on which track provision is made for grade separations for trains, and likewise for vehicular traffic.

GEOLOGY

Thirty-six miles in a northeastern direction from San Francisco, Calif., near the upper end of San Francisco Bay and connected to it by a narrow strip of water known as Carquinez Strait, is a body of water about 31 000 acres in extent. It is formed by the united waters of the Sacramento and San Joaquin Rivers flowing westward toward the Pacific Ocean and being retarded in their course by a range of mountains known as the Coast Range. The Carquinez Strait forms a connecting link by which the rivers unite with San Francisco Bay (see Fig 1). This inland body of water is known as Suisun Bay and, in former ages, covered a much larger area. After filling in the large inland areas now known as the Sacramento and San Joaquin Valleys, the two silt-carrying rivers began to deposit their sediments against the Coast Range, thereby reducing the area of this ancient sea and building up adjacent alluvial lands which are not equalled in fertility

In some cases inland bays along the Suisun shore line were filled with sediments containing a large percentage of vegetable matter. These bays are now virtually peat bogs, unstable in character. Such material is also found on the bottom of Suisun Bay.

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[1] Presented at the meeting of the Structural Division, Sacramento, Calif., April 24, 1930.

[2] Chf. Engr., Northwestern Pac. R. R.; Engr., M. of W. and Structures, S. P. Co., Pacific Lines, San Francisco, Calif.

Geologically, some interesting events have occurred in this vicinity. Evidence is found of former recurrent elevation and subsidence of the surface; pronounced earthquake faults exist upon which adjacent blocks of the earth's crust have moved in a northern and southern direction; and tremendous resistance to such movements has resulted in folds and distortion of the rock strata.

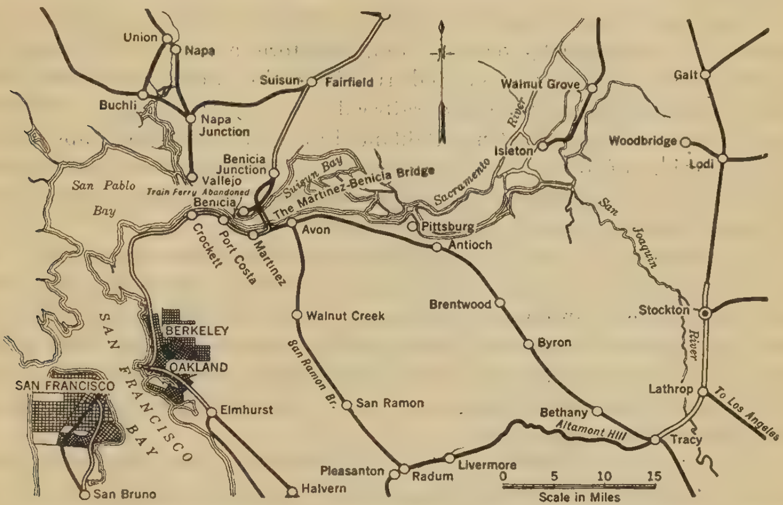

Union
Napa
Suisun
Fairfield
Walnut Grove
Galt
Buchli
Napa Junction
Woodbridge
Lodi
River
Isleton
Sacramento
Benicia Junction
Vallejo
Suisun Bay
The Martinez-Benicia Bridge
San Pablo Bay
Train Ferry Abandoned
Benicia
Pittsburg
Avon
Antioch
Crockett
Port Costa
Martinez
San Joaquin
Brentwood
Stockton
Walnut Creek
BERKELEY
OAKLAND
SAN FRANCISCO BAY
River
Byron
SAN FRANCISCO
Lathrop
To Los Angeles
Elmhurst
San Ramon Br.
San Ramon
Bethany
Altamont Hill
Tracy
Livermore
Pleasanton
Radum
Halvern
San Bruno
0 5 10 15
Scale in Miles

FIG. 1.—MAP SHOWING LOCATION OF THE MARTINEZ-BENICIA BRIDGE.

The hillsides bordering along the Carquinez Strait are broken and disintegrated to such an extent as to be very unstable, and slides are frequent, entailing unusual care and extra construction on the part of those engaged in the building and maintenance of railroads and highways in this territory.

EARLY RAILROAD DEVELOPMENT

The first railroads in California were constructed out of Sacramento, the first line being to Folsom. This was followed by the completion, in May, 1869, of the famous Central Pacific Railroad that formed a link in the first transcontinental line.

During the construction of the Central Pacific, a railroad was built in 1868 from South Vallejo to Sacramento, *via* Suisun, passing over a spur of the Coast Range and having a summit elevation of 315 ft. above sea level. In connection with this railroad a steamer line was operated from South Vallejo to San Francisco.

Next followed the construction, in 1869, of a railroad from Sacramento to Oakland, *via* Tracy, passing over the Altamont Summit of the Coast Range at Elevation 740 above sea level.

The railroad from Oakland to Martinez, along the southern shore of San Francisco Bay and the Carquinez Strait, was placed in operation in January, 1878, and, in the latter part of 1879, was also extended to Tracy. Thereupon, the steamer line to San Francisco was discontinued, and a ferry was established across the Carquinez Strait, connecting South Vallejo with the Tracy-Martinez-Oakland Line.

In December, 1879, a more direct low-level route between San Francisco and Sacramento being desired, the railroad passing through Suisun was projected directly across the marsh and peat lands on the north shore of Suisun Bay to Benicia.

A bridge across the Carquinez Strait or Suisun Bay was considered, but was found not economically feasible, and, therefore, a train-ferry system was established by which passenger and freight trains were transported from Benicia to a connection with the railroad on the south shore, previously referred to as the Tracy-Martinez-Oakland Line. This was accomplished by the construction of docks and car-ferry slips at Port Costa and also at Benicia, and by building the steamer, *Solano,* which remained in service until 1930 after fifty-one years of operation.

It is interesting to note that the designer of this boat made use of the principle of the truss bridge in designing the hull. In 1879, trains were small and rolling equipment was light; but notwithstanding this the *Solano* remained the largest car-ferry in the world up to the time it was removed from service. In 1914, as part of a second-track program, double-track car-ferry slips were constructed, and a second car-ferry, the *Contra Costa,* was put into commission.

It is interesting to note that the docks and slips were maintained with untreated piling from 1879 to 1919, a long life indeed for such original construction. However, in 1919, an infestation of marine borers (*teredo navalis*) destroyed the piling, and since that time creosoted piling has been used. The presence of the marine borers was due to the encroachment of salt water up the rivers, which, in turn, was caused by the use of fresh water for irrigation and by a long series of years in which the rainfall was sub-normal, thereby reducing the flow of fresh water through the strait.

The bridging of Suisun Bay or Carquinez Strait had long been under consideration. In the late Eighties newspaper discussions of the project were published and the Army Point-Suisun Point alignment was seriously considered by railroad officials. Later, a prominent bridge engineering company was requested to make studies, and following this, in 1904, during the time of the late E. H. Harriman, the late William Hood, M. Am. Soc. C. E., prepared plans for a bridge to replace the ferry system. Sundry locations were investigated, including one some distance up stream from the Army Point-Suisun Point alignment, and also several in the vicinity of Benicia and Port Costa, and others as far west as Vallejo Junction.

A series of test piles driven across the Bay showed that the bottom was of soft material, and that the depth to hardpan and rock strata was great; but it gave no clue as to the exact character of the bed-rock. It was decided to build a low-level bridge consisting of a swing bridge with a certain number of fixed spans and ballast deck trestle approaches. However, the application was denied by the War Department and the project was then dropped for the time being. Subsequently, the bridge studies were resumed and a crossing near Antioch was considered in connection with the extension of the Walnut Grove Line, but this route was rejected.

All routes crossing the Carquinez Strait proper were then likewise rejected because of the broken formation, slips in the hillsides, the nature of the terrain, the great depth of water, and, most important, the unfortunate necessity of crossing a well-established earthquake fault known as the "Southhampton."

Finally, in 1927, traffic conditions became acute and the Company faced the necessity of replacing the two existing car-ferries and buying a third, or of building a bridge. The bridge was then an economic necessity.

The two solid rocky points of land—one known as Army Point on the north shore of Suisun Bay, two miles east of Benicia, and the other as Suisun or Bulls Head Point near Martinez and adjacent to the tracks of the Tracy-Martinez-Oakland Line—offered the necessary supporting ground for the approaches and abutments, and permitted the solution of the railroad problem of providing connecting lines.

The Government demands a navigable channel 300 ft. wide for deep-draft vessels near the Martinez shore, but small craft, including fishing boats, use the entire width of the Bay. The presence of considerable soft mud on the bottom was known and the existence of hard rock below the mud was surmised; but the nature, extent, and depth below the water surface of such rock was not known. Proper data of the geological and earthquake fault condition were not of record. Accordingly, after a preliminary survey and a paper location of the route had been made, the services of geologists of experience and reputation were secured to report on the geological features, particularly with reference to the earthquake faults. This was followed by an exact determination of bed-rock conditions. For such purpose a row of diamond drill holes was put down across the Bay on the alignment of the bridge, as then conceived.

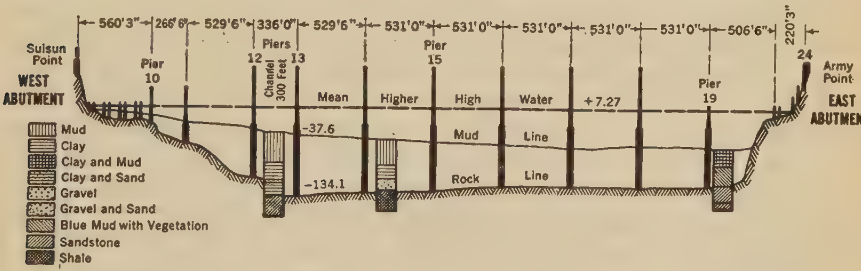

560'3"
266'6"
529'6"
336'0"
529'6"
531'0"
531'0"
531'0"
531'0"
531'0"
506'6"
220'3"
Suisun Point
WEST ABUTMENT
Pier 10
Piers 12 13
Channel 300 Feet
Pier 15
Pier 19
24
Army Point
EAST ABUTMENT
Mean Higher High Water +7.27
-37.6
Mud Line
-134.1
Rock Line
Mud
Clay
Clay and Mud
Clay and Sand
Gravel
Gravel and Sand
Blue Mud with Vegetation
Sandstone
Shale

FIG. 2.—CROSS-SECTION OF SUISUN BAY ALONG CENTER LINE OF BRIDGE.

Bed-rock, consisting of alternate layers of shale and sand stone tilted up on edge at an angle ranging from 60° to 70° from the horizontal, was found at varying depths extending across the channel (see Fig. 2). Drill holes were carried well below the rock surface and the cores were carefully preserved and recorded.

A standard well rig, supported on a 14-pile staging, was used in drilling through the mud. A steel frame of angle irons was lowered through the water to the mud surface and through this a 10-in. well-casing extended to bed-rock, after which the diamond drill was used to explore the rock. A 2-in. rock core was secured. The nature of the mud and gravel overlying the rock was studied, and bearing values were determined. It was thus concluded that the bridge piers should be carried to, and founded on, the bed-rock.

The depth below low water to bed-rock ranges from 115 ft. near Army Point to 136 ft. near Suisun Point, at the location of the lift span. From these depths the rock slopes rapidly upward to the shore line on both sides. The depth of mud near Army Point is 59 ft., while the greatest depth of 101 ft. is south of Pier No. 13. The greatest depth of water is 57 ft. on the Army Point side. It is interesting to note that before the deposition of the present sediments the eroded channel was on what is now the shallow side.

The geologists reported the existence of two well-established major faults extending, parallel to each other, in a northwestern and southeastern direction. The "Southhampton" Fault is west of the bridge and passes under the ferry lanes between Benicia and Port Costa, and the "Green Valley" Fault is east of the bridge, 5 or 6 miles away.

In the Martinez hills various fault lines run in a direction angular to the major faults. The dip of the strata showed a reversal of direction, thereby indicating a folding of the earth's crust, local in extent, and also the development of a syncline. The direction of one of these fault lines was such as to present a serious question. Did it pass under the bridge? Unfortunately, the sediments in the Bay hid its true direction.

While there was some difference of opinion on the part of the geologists, they all agreed that the fault was not active and that any future movement of adjacent blocks along this fault line would be nominal. Its projection on a map shows that it might pass across the bridge tangent near the Army Point shore. On the other hand, the evidence showed that the fault might be a local shear or break in the formation caused by the folding of the strata, but which might not extend far into the formation upon which the bridge was located. This formation on the east was classified as "Chico-Cretaceous" and bedded against it on the west was a formation known as "Martinez," and bedded on the "Martinez" was a formation known as "Tejon." The folded and distorted strata seemed to be confined to the "Martinez" and "Tejon" formations.

However, if the fault line did extend beyond Army Point, it became important to determine its exact location, or at least to avoid the possibility of the most northerly deep-water pier (No. 19, Fig. 2) being placed on it. It was evident from the geological data developed that any slippage along the so-called Martinez Fault would cause the "Martinez" formation to be projected northward into and across the alignment of the bridge. It was decided, therefore, to put down additional diamond drill holes on the site of Pier No. 19. These core drillings, as well as those of all other diamond drill holes, were carefully studied by the geologists and paleontologists. Microscopic and other studies were made to determine from the fossils whether there had been any change in the rock formation. No such change was found, and the conclusion followed that the Martinez Fault did not extend across the bridge location and that the bridge foundations would rest solely on one monolithic block of geological formation, namely, the "Chico-Cretaceous."

However, for other obvious reasons, it was decided to design the bridge piers to resist earthquake vibration. Accordingly, they were designed with centers of gravity of mass as low as possible. Concrete specified for this purpose had a compressive strength of 2 500 lb. per sq. in., and a tensile strength of 400 lb. at the end of 28 days. It was further specified that the piers should actually be founded on bed-rock and that they should have 30 lb. per cu. yd. of steel reinforcement, placed so as to withstand earthquake shocks of an acceleration of 5 ft. per sec. per sec., which is twice the intensity of the San Francisco earthquake of 1906, and 1.5 times the intensity of the Tokyo, Japan, earthquake of 1923.

The superstructure was designed to resist similar shocks, the tops of the piers being widened and the base of the steel bridge seats being notched into the concrete in order to prevent the spans being pulled off their piers. In designing a pier structure of this kind to resist earthquake vibrations, a quick reduction from a large mass to a smaller mass is a source of weakness; likewise, a construction joint is undesirable and monolithic construction is essential. To secure this, as between caisson and pier shaft, and to overcome the difference of inertia of the two different masses, reinforcement rods extend upward from the outer walls of the caisson, pass through the distributing block, and curve inward and upward into the shaft of the pier (see Fig. 3).

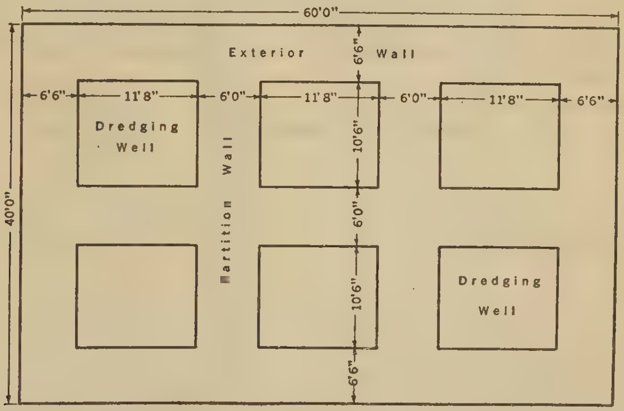


FIG. 3.—DIMENSIONS OF PIER NO. 13, MARTINEZ-BENICIA BRIDGE.

BRIDGE FOUNDATIONS

The substructure consists of two U-abutments, eight viaduct pedestal piers on land, and fourteen viaduct pedestal piers and ten truss span piers in water. Of the ten piers in water, eight are classified as deep-water piers, inasmuch as the rock bottom upon which they are founded ranges in depth below low water from 115 to 134 ft.

Pier No. 11 represents the shallow-water type that can be built by the open coffer-dam method. The size of the concrete base from which the pier shaft rises is 38 by 60 ft. It is oval in shape, with the top of the coping 16 by 49 ft., the side batter 24 on 1, and it was constructed within a steel sheet-pile coffer-dam, rectangular in shape.

Pier No. 13 is the deepest of the deep-water piers. The caisson for this pier was 40 to 60 ft. in cross-section and contained six dredging wells, 10.5 by 11.7 ft. in cross-section (see Fig. 4). The exterior walls were 6.5 ft., and the partition walls 6 ft. thick. The bottom of the caisson was of structural steel, shaped on the outside into a sharp cutting-edge, the interior partition struts conforming to the walls of the six dredging wells, being blunt on the lower surface to lend stability to the caisson during sinking operations

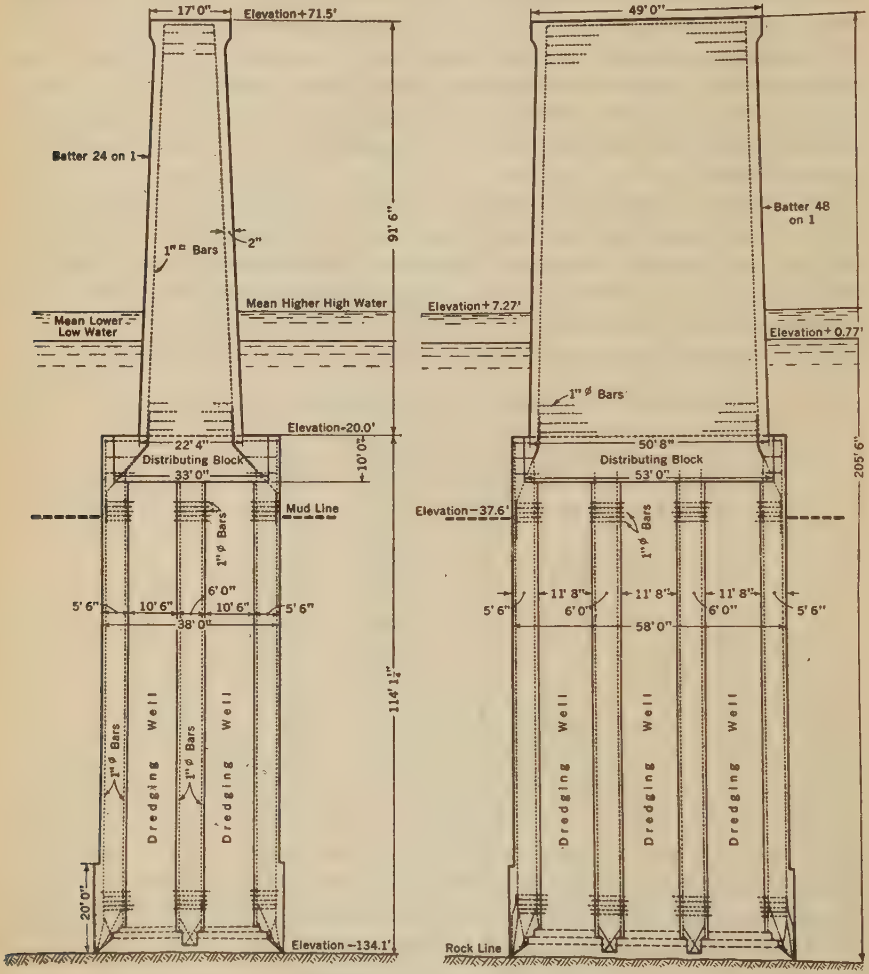

17' 0"
Elevation+71.5'
Batter 24 on 1
1"□ Bars
2"
91' 6"
Mean Higher High Water
Mean Lower Low Water
Elevation-20.0'
22' 4"
Distributing Block
33' 0"
10' 0"
Mud Line
1"⌀ Bars
6' 0"
5' 6"
10' 6"
10' 6"
5' 6"
38' 0"
114' 1¼"
1"⌀ Bars
Dredging Well
1"⌀ Bars
Dredging Well
20' 0"
Elevation -134.1'
49' 0"
Batter 48 on 1
Elevation+7.27'
Elevation+0.77'
1"⌀ Bars
50' 8"
Distributing Block
53' 0"
205' 6"
Elevation-37.6'
1"⌀ Bars
5' 6"
11' 8"
6' 0"
11' 8"
11' 8"
6' 0"
5' 6"
58' 0"
Dredging Well
Dredging Well
Dredging Well
Rock Line

FIG. 4.—CROSS-SECTION OF CAISSON, PIER NO. 13.

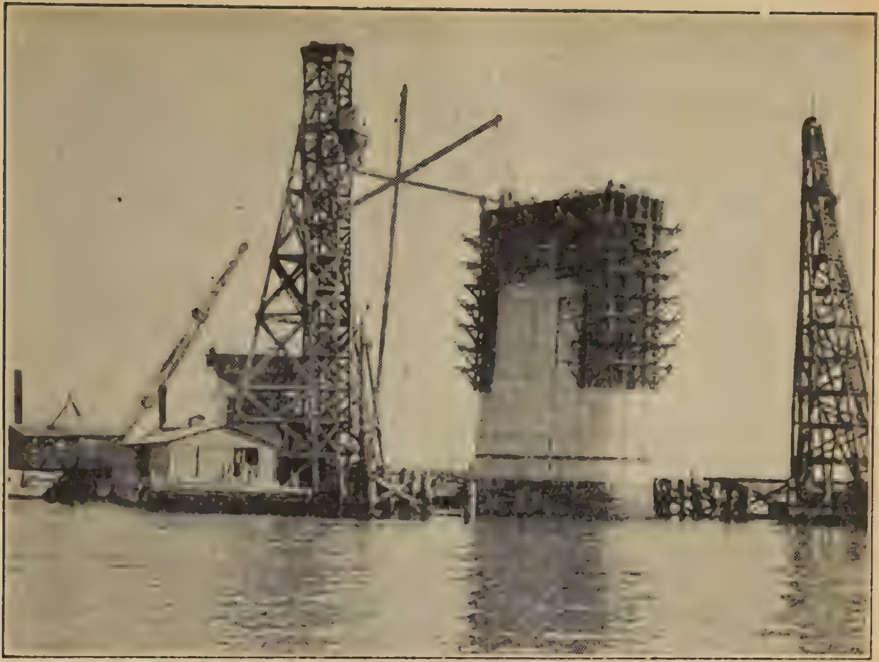


FIG. 5.—PIER NO. 12, MARTINEZ-BENICIA BRIDGE, FORMS SET TO FINISHED HEIGHT.

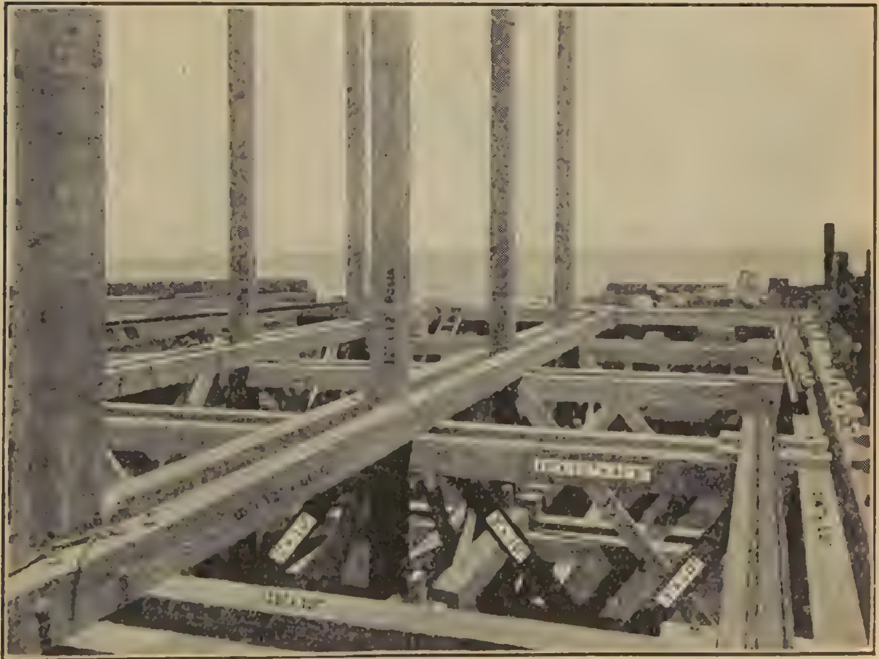


FIG. 6.—TYPICAL COFFER-DAM BRACING, PIER NO. 11.

and to distribute the foundation reactions better. Except for the dredging wells, the caisson distributing block and pier shaft were of reinforced concrete. The dredging wells were filled with tremie concrete. The total volume of concrete in this pier is 12 283 cu. yd. The total weight of structural and reinforcing steel is 444 520 lb. Pier No. 12 (Fig. 5) is identical in design, except as to height. These two piers support the lift span and towers, and the adjacent spans.

The other deep-water piers are similar in design, except that the caisson dimensions are 38 by 60 ft. and the dimensions on top of the coping are 16 by 49 ft.

CONSTRUCTION

The construction of the part of the substructure on land followed the usual method of excavating to suitable rock foundation, placing forms, and pouring concrete; and the viaduct pedestals in water were constructed by driving coffer-dams of sheet-piling.

Piers Nos. 10 and 11 were constructed by the open coffer-dam method. Interlocking steel piles, 65 ft. long, were driven within timber falsework of piling on 7-ft. centers. Battered brace piles also were used, with two rows of 12 by 12-in. timber wales bolted to them. A single row only of sheet-piling was used in Pier No. 11. The inside bracing consisted of a crib or grillage of large, full length timbers, formed of 12 by 12-in. struts, 12 by 12-in. posts, and 12 by 24-in. wales tied together by 3 by 10-in. diagonal bracing, all securely bolted together (see Figs. 6 and 7).

The grillage was sunk nearly to bed-rock by being loaded with about 270 tons of steel rails. Mud and clay were dredged by means of a floating derrick barge, from which a clamshell bucket was operated.

When shale, sandstone, and bed-rock was exposed, about 1½ ft. of it was removed. The surface was then flushed clean, and concrete was poured "in the dry," the forms first having been set up inside the wales and sheet-piling. A mat 6 ft. thick was thus poured, after which the forms were set in to the prescribed section of 58 by 38 ft. and carried to Elevation — 36, where they were again set in to 53.4 by 22 ft. and carried to Elevation — 8. Above this elevation a pier of prescribed dimensions was constructed to bridge-seat elevation. One blow-out occurred while the coffer-dam was being unwatered, but the contractor overcame the difficulty without undue delay.

The construction of the deep-water piers presented a difficult problem. The depth of the water prevented open coffer-dam work, or the use of the pneumatic process. It was proposed, therefore, to sink caissons by the open-well dredging method. The depth of water, the existence of a maximum tidal fluctuation of about 10 ft., the swiftness of the current during the winter floods and at rip tide, and the softness of the underlying bay deposits, all made the launching and sinking of a timber crib difficult.

The contractors, to whom the foundation contract was awarded, proposed a unique method of sinking these deep-water caissons. The accepted construction involved, first, the driving of piles to support an octagonal-shaped platform. Upon this platform eight gallows frames equipped with hand

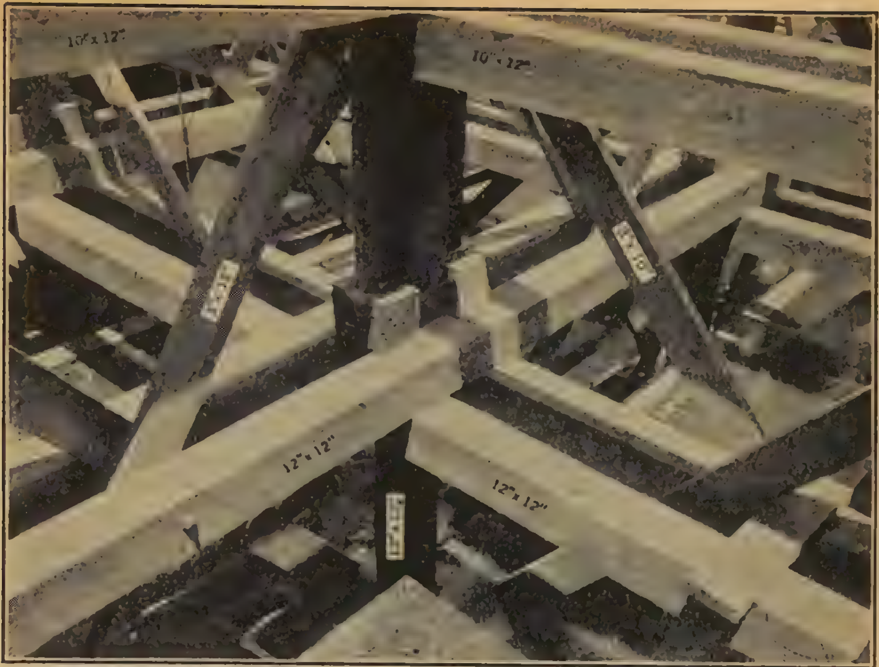

10"x 12"
10"x 12"
12"x 12"
12"x 12"

Fig. 7.—Typical Coffer-Dam Bracing, Pier No. 11, Martinez-Benicia Bridge.

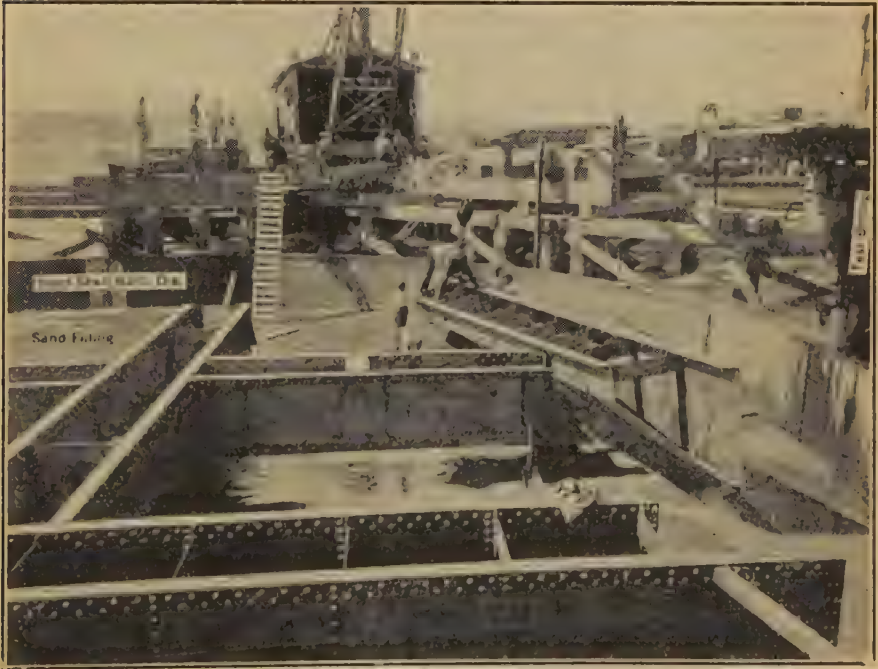

Sand Filling

Fig. 8.—View of Erection of Cutting-Edge of Caisson, Pier No. 12.

winches and with cables passing over sheaves set at the top of the tower were erected. By this apparatus a steel shell that may be likened to a bottomless oil tank, 81 ft. in diameter, was erected and lowered through the water into the mud, which it penetrated to a depth of 15 to 30 ft., depending on the compactness of the material. This shell was built up of circular sections, 10 ft. in height, which were bolted together, permitting the later removal of all but the part in the mud.

The top steel shell was above high water and was filled with suitable sand dredged from up the river. The area of land thus created permitted the exact staking out and fabrication of the steel cutting-edge of the caisson that was then sunk to rock (see Fig. 8).

After the steel cutting-edge had been erected, the reinforcement steel was placed and the concrete poured in successive lifts of 10 ft. in height. All this work was done above water, which permitted the caisson to be constructed of reinforced concrete of the desired quality and strength. Exact centering and orientation was thus assured.

When the first 20 ft. of concrete shell was completed, the sand was excavated from the six dredging wells by clamshells operated from floating derrick barges. The caisson gradually settled into the sand deposit and the underlying strata, as the excavation from the dredging wells continued.

At the time the concrete caisson was started, four well holes were put down within the steel shell, by which the exact elevation of the bed-rock was determined at four points, conforming to the four sides of the caisson. This information determined the proper height of the caisson prior to sinking and also the elevation at which to begin construction of a timber coffer-dam that extended from Elevation —20 to well above high water.

This coffer-dam (constructed of 12 by 12-in. timbers, drift-bolted, and properly caulked) was removable; temporarily, it rested upon, and was connected to, the concrete caisson by suitable tie-rods. The coffer-dam permitted pouring the distributing block (Fig. 3) and erecting the pier shaft from a depth 30 ft. below high water. It also effected a saving in concrete by the reduction of the large section of the caisson to the smaller section of the pier shaft, and created a temporary bridge seat or shelf which was used to support the steel bents of a falsework bridge span.

When the caisson was landed at or near bed-rock, and after the greater part of the material immediately above bed-rock was removed by hydraulic jetting and clamming, a deep-sea diver explored the bottom completely, telephoning the conditions existing within and under the caisson. He particularly noted the relation of the cutting-edge to bed-rock, and the amount of excavation still to be done.

In all cases considerable clay or gravel was found compacted under the partition walls and banked up against the cutting-edge. By means of the information thus furnished and by successive examinations the contractor was enabled to direct his jetting operations until the foundation area was reported satisfactory and in a clean condition.

The diver was able to supply accurate information as to the penetration of the cutting-edge into the foundation rock and in each case he brought up samples of the rock encountered. This was perhaps the first time a deep-sea diver was used at such great depth of water in open caisson work. It enabled the railroad engineers to verify the diamond drill borings, to know definitely the nature of the bed-rock formation, and to be assured that the bridge piers were actually resting directly upon the rock. Work was continued until a full bearing was secured and an interlocked connection established between the concrete steel cutting-edge and bed-rock.

Thereafter, the pouring of the concrete seal, consisting of tremie concrete, began. This operation consisted of lowering a tremie pipe, 12 in. in diameter, sometimes to a depth of 150 ft. The upper part consisted of a funnel-shaped hopper and the entire pipe was supported by a line leading to a hoisting engine, so that it could be lowered and raised as desired. A plug of excelsior was placed in the tremie pipe and the pipe was filled with concrete, about 4 cu. yd. being required.

Thereafter, the column of concrete was not permitted to break and as it was deposited in the hopper the pipe was raised a few inches to permit the concrete to mushroom out at the bottom. The process continued without interruption until the dredging well was sealed to the desired height, and, successively, the six dredging wells were thus filled. After a sufficient quantity of concrete was in place the lower end of the tremie pipe remained from 6 to 10 ft. below the surface of the concrete during the pour.

At the beginning of the work, in order to study the condition of the tremie concrete, and to assist in visualizing the interior of the dredging wells, the tremie concrete was first brought up to an elevation 30 ft. above bed-rock. After the concrete had hardened the water was pumped from the caisson, the sides were washed down, and the mud and laitance were removed, this deposit being usually about 1½ ft. thick. Samples were then broken out of the tremie concrete and tests made. In all cases a most excellent quality of concrete was found. Subsequently, being assured of the results obtained, the engineers permitted the tremie concrete to be brought, at one pouring, to near the elevation of the distributing block.

Within the recessed walls at, and resting upon, the prepared surface of the top of the caisson and the tremie-filled dredging wells, the distributing block was poured next. This was a large mass of reinforced concrete, 10 ft. thick, 50 ft. long, and 33 ft. wide (see Fig. 3).

The pier was completed by pouring the shaft to bridge-seat elevation in 10-ft. lifts. The wooden forms for this purpose were held in position by being bolted to the concrete section that was immediately below the section being poured.

The removal of the octagonal falsework, the pulling of the piling, the removal of the steel shell, and the dismantling of the timber coffer-dam followed in sequence after the pier was well above water. Such salvaged material was used again as work began on the next pier to be constructed. Fig 3, showing Pier No. 13, is typical.

The construction methods proved highly successful. In all cases the piers landed on bed-rock with only slight variation from the true center and inclination from the vertical. For instance, the caisson of the deepest pier

(No. 13) was off from true orientation by a southerly departure of 0.73 ft. and an easterly departure of 0.89 ft. The inclination from the vertical was 0.75 ft. in a height of 114.1 ft., which is equal to a batter of 150 on 1. Of course, even this slight error was not carried upward, as forms of the pier shaft were set to instrumental exactness.

Pier No. 12 may be used to illustrate the most accurate condition of placing. In this case the caisson departed from true orientation to the amount of 0.2 ft. northerly and 0.3 ft. westerly, the inclination from true vertical being only 0.14 ft. in 86.4 ft., which is equivalent to a batter of 600 on 1.

When one considers that these more or less unwieldy columns of concrete, with base areas of 2 400 sq. ft., and having a height in the one case equivalent to that of a 9-story office building, and weighing about 19 000 tons, have been set to the aforementioned accuracy, under the conditions found to prevail in connection with this work, the result appears to be marvelous.

To illustrate some of the construction difficulties the fact may be mentioned that at one time the caisson of Pier No. 12, was found to be out of level 1.3 ft. However, by confining the dredging to the wells on the high side, it was gradually brought back to a level position.

At another time when the steel shell was being set and sand was being placed at Pier No. 19, difficulty was encountered due to the tendency of the steel shell to settle to the north because the underlying strata was soft tule mud. The contractor righted the shell successfully, but due to the further settlement an additional 10-ft. ring of steel had to be applied. When next he attempted to fill the shell with sand, further settlement occurred on one side and the surrounding mud bulged upward.

Sometimes the caisson would fail to move downward, and as it was desired not to carry the dredging too far below the cutting-edge, small charges of dynamite were set off at the bottom of the dredging wells. These had the effect of causing the caisson to move. These instances illustrate the flexibility of the adopted method of sinking caissons.

SUPERSTRUCTURE

Fig. 9 is a view of the finished bridge, which is 5 603.5 ft. between abutments, accommodating two tracks spaced on 13-ft. centers. The elevation of the base of rail is 85.6 ft. above railroad datum, which provides a vertical clearance above mean higher high water of 70 ft. for the entire navigable distance of 4 050 ft. across the Bay. Provision also is made for a 328-ft. vertical lift span that will, when raised, clear mean higher high water by 135 ft. (see Fig. 10). The foregoing clearances are increased by 6.5 ft. when the tide is at mean lower low water.

The bridge is on tangent, except that spiraled 4° curves at each end extend outward on the viaduct sections. A length of 4 556.5 ft. of the bridge is on level grade, with approach grades of 0.45% on the north and 0.66% on the south end.

The superstructure consists of seven through truss spans of 526 ft. each, the distance center to center of piers being 531 ft., and also one vertical lift span,

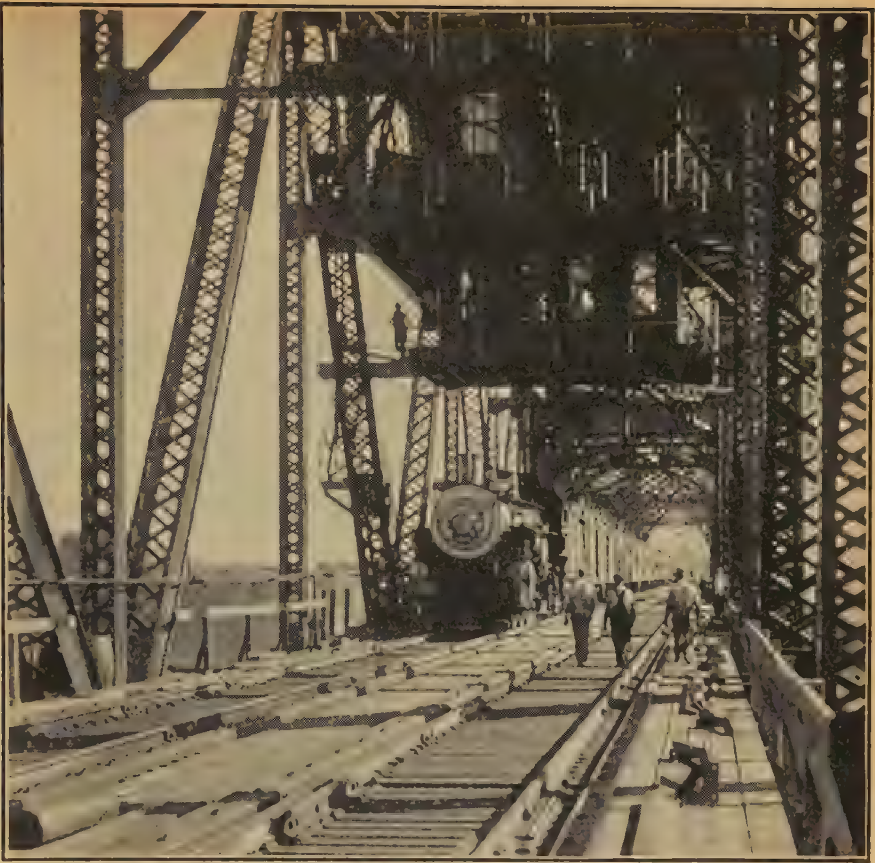


FIG. 9.—VIEW OF THE MARTINEZ-BENICIA BRIDGE, WITH TRAIN ON SPANS 12 AND 11.

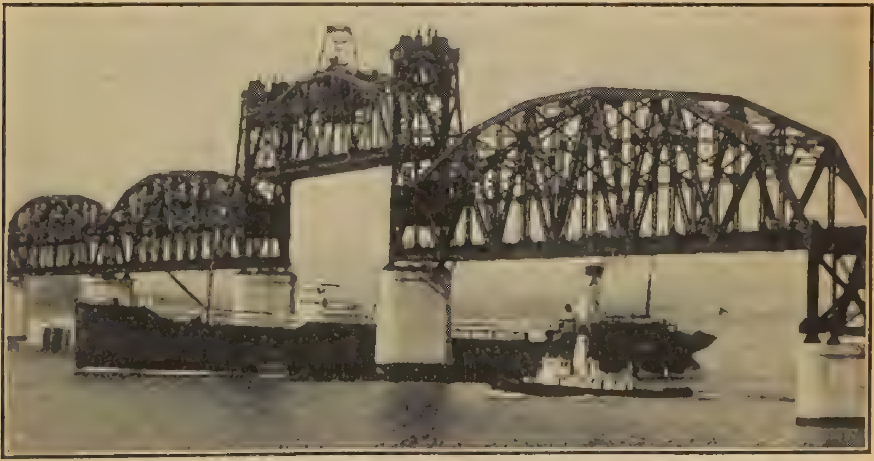


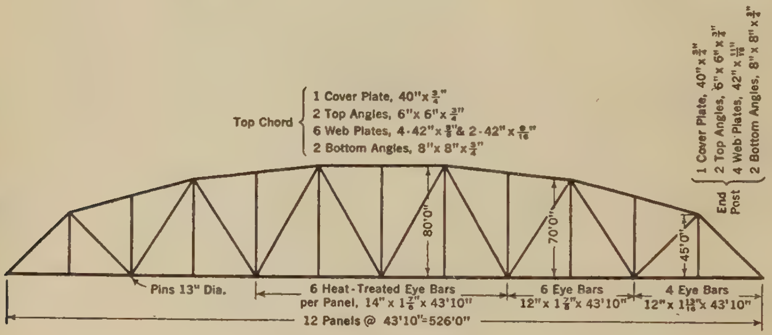
FIG. 10.—VIEW OF LIFT SPAN IN RAISED POSITION, MARTINEZ-BENICIA BRIDGE.

328 ft. long; one deck span, 504 ft. long; one deck span, 264 ft. long; a viaduct approach on the south end of five 80-ft. girders and four 40-ft. tower girders; and on the north end a viaduct approach of one 100-ft. girder and three 40-ft. tower girders.

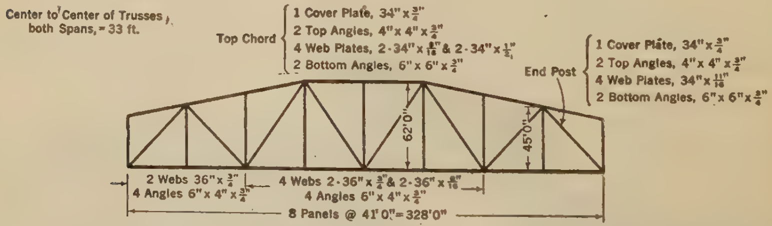
Truss-span lengths of 526 ft. are not necessarily the most economical, but navigation interests are better served by spans of 526 ft., primarily to permit the passage of long barge tows without the danger of side-swiping of piers. After conference with the engineers of the War Department and navigation companies, the 526-ft. length was agreed upon.

The towers for raising the lift span are integral with the spans adjacent to the lift span and rise to a vertical height of 139 ft. above the bridge seat. The viaduct towers and girders are of the usual standard design.

Design.—The superstructure was designed for a minimum live load on each track equivalent to Cooper's E-90 followed and preceded by a uniform load of 7 500 lb. per lin. ft. of track. The uniform load was used or omitted at either or both ends of the concentrated load to give maximum stress.


Top Chord
1 Cover Plate, 40"x 3/4"
2 Top Angles, 6"x 6"x 3/4"
6 Web Plates, 4-42"x 5/8"& 2-42"x 9/16"
2 Bottom Angles, 8"x 8"x 3/4"
End Post
1 Cover Plate, 40"x 3/4"
2 Top Angles, 6"x 6"x 3/4"
4 Web Plates, 42"x 11/16"
2 Bottom Angles, 8"x 8"x 3/4"
80'0"
70'0"
45'0"
Pins 13" Dia.
6 Heat-Treated Eye Bars per Panel, 14"x 1 7/8"x 43'10"
6 Eye Bars 12"x 1 7/8"x 43'10"
4 Eye Bars 12"x 1 13/16"x 43'10"
12 Panels @ 43'10"=526'0"

526 - FT. FIXED SPAN


Center to Center of Trusses, both Spans, = 33 ft.
Top Chord
1 Cover Plate, 34"x 3/4"
2 Top Angles, 4"x 4"x 3/4"
4 Web Plates, 2-34"x 9/16"& 2-34"x 1/2"
2 Bottom Angles, 6"x 6"x 3/4"
End Post
1 Cover Plate, 34"x 3/4"
2 Top Angles, 4"x 4"x 3/4"
4 Web Plates, 34"x 11/16"
2 Bottom Angles, 6"x 6"x 3/4"
62'0"
45'0"
2 Webs 36"x 3/4"
4 Angles 6"x 4"x 3/4"
4 Webs 2-36"x 3/4"& 2-36"x 9/16"
4 Angles 6"x 4"x 3/4"
8 Panels @ 41'0"=328'0"

328 - FT. LIFT SPAN

FIG. 11.—DIAGRAM SHOWING PRIMARY DIMENSIONS OF LIFT AND MAIN FIXED SPANS.

This loading is 50% heavier than the present standard and with it stresses one-third higher than standard are permitted. In calculating maximum stresses due to live load and centrifugal force in girders and trusses of spans carrying two tracks, both tracks were assumed to be loaded simultaneously and 90% of the specified live load was used.

Silicon steel was used for the principal members fabricated from shapes, and carbon steel for the remainder, except that the tension bars in the lower chord, consisting of eye-bars, were heat-treated. The total weight of the

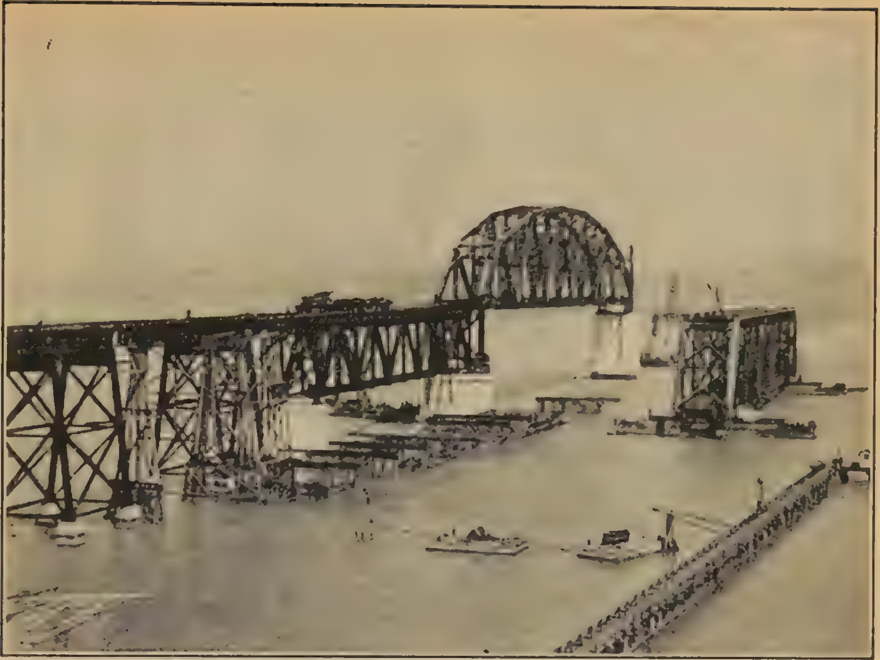


FIG. 12.—FLOATING SPAN 19 FROM FALSEWORK TRESTLE, MARTINEZ-BENICIA BRIDGE.

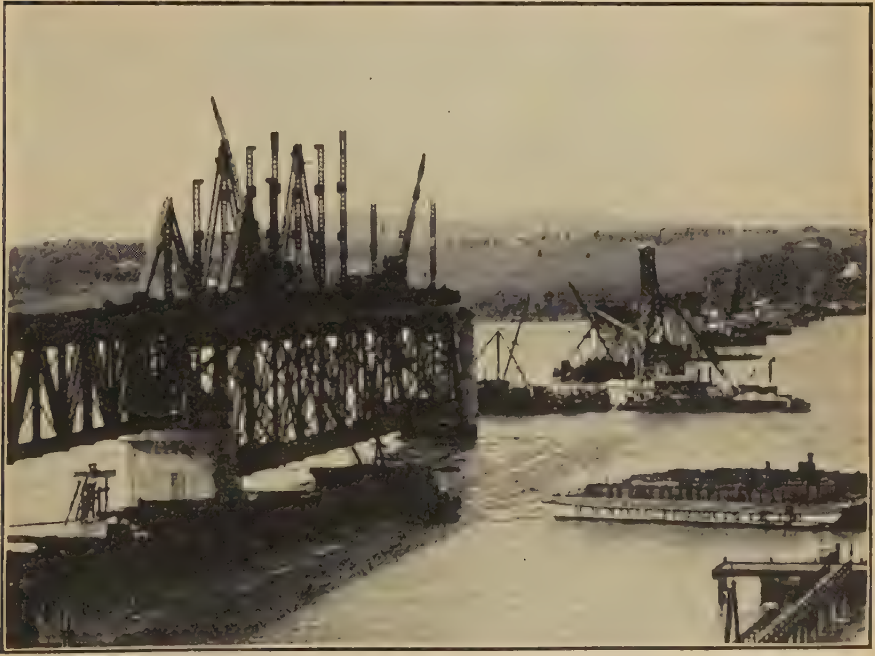


FIG. 13.—MARTINEZ-BENICIA BRIDGE: ERECTION OF SUPERSTRUCTURE AND CONSTRUCTION OF PIERS.

structural steel was 44 000 000 lb., comprising 25 000 000 lb. of silicon steel, 5 500 000 lb. of heat-treated eye-bars, and 13 500 000 lb. of carbon steel. The permissible unit stresses as used in the design were, as follows:

For axial tension in net section:

Carbon steel	21 300 lb. per sq. in.
Silicon steel	32 000 lb. per sq. in.
Heat-treated eye-bars	36 000 lb. per sq. in.

For axial compression in gross section:

Carbon steel.... $20\,000 - 70\frac{l}{r}$ (up to 16 500 lb. per sq. in.)

Silicon steel.... $30\,000 - 105\frac{l}{r}$ (up to 24 750 lb. per sq. in.)

The primary dimensions of the superstructure are shown in Fig. 11. The lower chords consist of heat-treated eye-bars; all other truss members are of rolled structural shapes. The lift span, shown in Fig. 10, is a riveted truss throughout.

CONSTRUCTION

Steel Erection.—Various methods of erection were considered, and it was deemed impossible to use timber piling successfully and economically owing to the depth of water, the softness and depths of the underlying mud, the swiftness of the current, and the height of the piers above water; nor was it deemed practicable to use the alternative method of "barging" the completed spans into position. An economical and satisfactory falsework method which avoided driving piling under the main spans, was adopted by the contractor. The steel was erected from the south or Martinez end, progressively working across the Bay to the Army Point end. The steel yard, therefore, was established at and near the south bridge-head, the railroad tracks, including the main-line tracks, being used for storage and switching purposes.

Proceeding from the abutment the tower posts and girders, comprising 560 ft. of viaduct, were erected by derrick car as far as Pier No. 10. Temporarily, the last pair of girders of the eastward track were used for a turnout approach to a falsework trestle that was driven immediately adjacent to and up stream above Pier No. 10 (see Fig. 12). This falsework trestle was provided for the erection of the falsework span, but the 264-ft. deck span was erected on it as a matter of convenience and was floated into its true position by means of barges and placed upon Piers Nos. 10 and 11. Next, Span 19, between Piers Nos. 19 and 20, was erected on the same falsework. This span, however, was used as falsework eight successive times in the erection of the remaining spans of the bridge. The procedure was as follows: After erection, Span 19 was floated from its falsework into position between Piers Nos. 11 and 12 (see Fig. 13) where it rested upon two temporary steel bents that extended to the shelf formed by the top of the caisson at Elevation — 20. These bents were secured to the pier shaft.

When the span had been set in place the elevation of its top chord was such that the necessary blocking to support the permanent span could be placed. Permanent spans were then built as follows: (1) Lay the floor system and lower chord; (2) erect the vertical posts, diagonal web members, and top chord; (3) jack up the completed span to release the falsework truss; (4) float the falsework span into position between the next piers; and (5) lower the completed span on its piers. To erect the lift span, the falsework span was placed back on the falsework bents and cut apart at a panel to make it of proper length to go between the piers.

After Span 18 was completed, the falsework span was finally floated into position as a deck-truss span for connection with the north viaduct approach.

Connecting Railroad Tracks.—The finished project included the construction of the equivalent of 20 miles of single track, and the reconstruction of 24 miles of branch line to main-line track.

On the north end the eastbound track descends on a 2.2% grade to a connection with the main line in a distance of 0.93 mile. This steep grade is permissible due to the fact that the train movement is descending from the high elevation of the bridge to the lower level of the existing main line, and a flatter grade is unnecessary and uneconomical. The reverse is true of the westbound movement.

The operating requirement is that the locomotive load be not reduced on full-tonnage trains moving from the north or east toward the west over the bridge. Therefore, it was necessary to build 3.64 miles of track on an ascending grade of 0.45% to attain the bridge elevation. This required an entirely new location, leaving the main line at Goodyear Station and developing a supporting grade line on the foothills to the north (see Fig. 14). The line crosses an arm of Suisun Bay, one of the inlets or marsh areas previously mentioned. The height of fill was 50 ft. and test wells proved the soil to be in the nature of peat, saturated with water, for a considerable depth. It was concluded, therefore, that it would be uneconomical to make an embankment, because the subsidence would be too great and the track unstable.

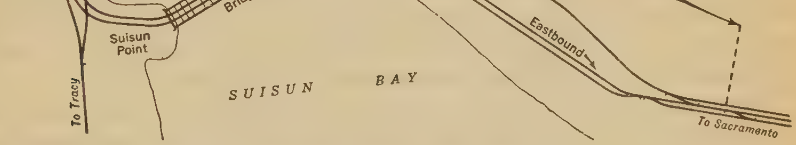


FIG. 14.—PLAN OF MARTINEZ-BENICIA BRIDGE AND APPROACHES.

The engineers were not without previous knowledge of the action of such marsh areas, inasmuch as the present main lines in this territory, for a distance of 10 miles, have been maintained under difficulties for years. The subsidence has been as much as 80 ft., even with the subgrade near the natural surface. Therefore, the choice was a steel viaduct, 1 860 ft. long. It consists of twenty-one 60-ft. girder spans and twenty 30-ft. girder tower spans, having concrete abutments and pedestal piers of concrete, each resting on 16 piles. The maximum length of piling was 80 ft. below the cut-off, which is below the water-table.

On the south end of the bridge the approach consists of a double track on a maximum 1% grade, extending in the direction of Oakland to Martinez Station. It was found impracticable, except at unjustifiable expense, to obtain an easier grade. However, 1% for eastbound traffic was not considered very objectionable as the grade would not restrict passenger trains unduly, and freights can be handled with "helper" engines, which will always be available, due to the large number of industries at Martinez requiring switching service.

Furthermore, most of the freight trains moving to and from San Francisco, and points to the south, will be handled over another approach, when the San Ramon Branch is reconstructed. This approach to the bridge will be on a 0.6% ascending grade eastward. Freight destined for Oakland (or the East Bay territory, as it is commonly called), and all passenger trains will still continue to move *via* Port Costa. A range of mountains extends in a north and south direction between the bridge and Oakland, making it necessary to follow the Bay shore on the north, or through Niles Canyon on the south, to reach either Oakland or San Francisco.

To facilitate such a movement, therefore, a second track, 4.4 miles long, has been constructed from the bridge to a connection with the existing double track at Port Costa. Part of this track is along the Suisun Bay shore and it has been riprapped as a safeguard against wave action.

The construction of the double-track aproach on the Martinez end involved some unusual problems. There, again, the engineers faced the problem of running track over marshland. The embankment was not as high as that on the north approach, however; the mud was shallower and below that there was hardpan. Other conditions also tended to make it desirable to construct a solid embankment. This embankment subsided during construction and the mud strata bulged outward, but excellent filling material was available economically and the job was accomplished without difficulty.

Miscellaneous Structures.—A concrete underpass was constructed through this embankment so that the westbound main line to Tracy could pass under the double track leading to the bridge. A 20-ft. vehicular subway was also constructed through the embankment.

Various other concrete structures, such as conduit boxes to handle numerous industrial oil pipes were constructed. Likewise, an overhead bridge was built to pass an industrial railroad over the main lines and sidings of the railroad (see Fig. 14). All structures were on pile foundations, driven into the mud strata to refusal, with the cut-off below water level.

The main lines are equipped with automatic block signals and in addition to the signals and interlocking plants an automatic train-stop system is installed on each side of the lift span. The control of the bridge-operating machinery, including the locks, rail locks, and signals, are handled by an electro-pneumatic interlocking machine with seven levers, provided with the customary mechanical approach and electric locking to insure that each function is operated in proper sequence and completely protected.

Each pier of the lift span is protected by a creosoted pile fender to permit the safe passage of sea-going vessels. This protection consists of a nose dolphin of fifty piles with two rows of pile spring lines leading to the piers. The spring lines are carried along the face of the pier by two rows of 12 by 12-in. timber fenders. The down-stream dolphin extends 46 ft. below the pier and the up-stream dolphin, 76 ft. above it. Protection is also provided at the other piers for smaller river craft.

All necessary navigation aids are provided and the top of the towers and the top chords of the bridge have flashing beacons for the benefit of air craft.

Concrete Tests.—Specifications for concrete required a strength of 2 500 lb. per sq. in. in compression and 400 lb. per sq. in. in tension within a 28-day period. After making exhaustive examinations of the available concrete aggregates, it was decided to require the cement to be fine enough that 85% would pass a 200-mesh sieve, and to use 3 lb. of diatomaceous silica per 100 lb. of cement, mixed with the cement at the cement plant. The concrete used in the piers, except the tremie concrete, was specified to be 1:2.5:5 mix by volume, or 1:2.4:4.6 by weight, with a water ratio of 0.9.

A well-graded crushed rock up to $2\frac{1}{2}$ in. in size, with sharp clean sand and 5 sacks of cement per yd. were used in this mix, and in the tremie concrete, $5\frac{1}{2}$ to $6\frac{1}{2}$ sacks per yd. were used, with $1\frac{1}{2}$-in. crushed gravel. The cement was placed in bins assigned to the particular job and after the tester approved the quality of the cement in it, the bin was sealed. Shipments to the job were made from this bin only.

Test cylinders were taken daily of the concrete poured. These were tested in the Railroad Company's laboratory for 7, 14, 28, 60 and 90-day curing periods. Greater strength was secured than was originally expected, as will be reflected by the following averages:

Age of test cylinders, in days	Ultimate compressive strength, in pounds per square inch
7	1 963
14	2 506
28	3 024
60	3 444
90	4 030

Because of the temperate climate, freezing and ice troubles were not encountered, but it was deemed necessary to consider the action of salt water on concrete. Therefore, that part of the concrete from a few feet below low water to a few feet above high water is formed of a rich 1:2:3 mix, using $1\frac{1}{2}$-in. rock aggregate. This concrete was poured dry and was subject to special spading and tamping so that a dense mixture was obtained. The surface was also treated to two coats of a coal-tar paint, selected after tests proved that the paint being used penetrated into the pores of the concrete.

Layout Work.—Ruling lines were established and caissons and piers were spotted by means of triangulation (see Fig. 15). First, a primary base line, 4 054 ft. long, was established and measured by an invar tape. From this a secondary base line was projected in a location near the south abutment, the triangulation stations of which were intervisible with other primary stations established on each side of the Bay. Another base line was established on the north side. Four triangulation stations, two on each side of the Bay, were thus established, by which all operations in connection with the sinking of the caissons and the erection of the piers were controlled with exact mathematical accuracy.

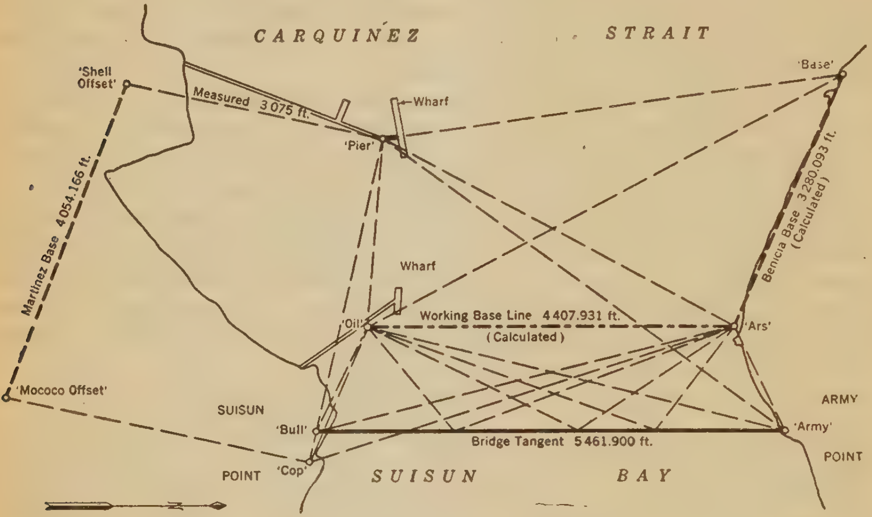

CARQUINEZ STRAIT
'Shell Offset'
Measured 3 075 ft.
Wharf
'Pier'
'Base'
Benicia Base 3280.093 ft. (Calculated)
Martinez Base 4054.166 ft.
Wharf
'Oil'
Working Base Line 4 407.931 ft. (Calculated)
'Ars'
'Mococo Offset'
SUISUN
'Bull'
ARMY
'Army'
Bridge Tangent 5 461.900 ft.
POINT
'Cop'
POINT
SUISUN BAY

FIG. 15.—BASE-LINE TRIANGULATION, MARTINEZ-BENICIA BRIDGE.

Grading Operations.—Extensive grading operations were required for the construction of the approaches. The contractor on the south end excavated and placed about 400 000 cu. yd. of material. The principal equipment consisted of two, 2-yd. steam shovels, operated by electricity, one 1-yd. drag-line, and one shovel operated by gasoline; and sixteen 10-yd. dump trucks, caterpillar tractors, and "bull-dozers." The contractor on the north end utilized dirt-moving equipment of ingenious design, his own development and manufacture. The principal equipment consisted of a 2-yd. Diesel steam shovel; 16-yd. heavy-duty dump carts and scrapers; and 60-h. p. caterpillar tractors. The dump carts were of all-steel welded construction; they were self-dumping and were mounted on caterpillar treads and hauled by caterpillar tractors. By means of this equipment about 500 000 cu. yd. of material was excavated.

CONCLUSIONS

The following unusual features of design and construction are worthy of special notice:

(1) The intensive examination into earthquake fault conditions and the design of bridge piers to resist earthquake shock;

(2) The creation of artificial islands of sand within steel shells by which the all-concrete caisson bridge piers were formed above water and sunk within the steel shells to bed-rock; and

(3) The temporary use of one of the permanent deck spans as falsework in the construction of the other bridge spans across deep water.

This bridge was placed in operation successfully on October 15, 1930. During the first 24 hours of operation the lift span was opened four times, and thirty-nine passenger and freight trains were handled over the bridge.

DISCUSSION

N. F. Helmers,[3] Esq. (by letter).—One of the outstanding features in the building of the foundations of this bridge was the constant use made of triangulation in measurements and layout. It is not unusual, of course, to resort to this method of locating piers for long crossings, but in this case a detailed system of signals was developed, permitting the use of triangulation to spot every pile in the falsework for each pier.

The base line was established approximately parallel to the bridge and ¼ mile down stream from the bridge site. From the two observation stations, all locations were given not only for the falsework, but for the sinking of caissons, erection of shaft forms, setting of anchor-bolts, etc.

In connection with pile-driving, signals to the driver were made by the use of a diamond-shaped target revolving about a center. One-half the diamond was white and the other half, red. The signaled direction to the pile-driver is indicated by the white half of the diamond target, which under ordinary visibility could be seen plainly without glasses, from any pier. For driving the falsework, the pile-driver was spotted at the intersection of two lines sighted from the two observation stations, the direction having been previously calculated to form such intersection for each pile as designated on the contractor's pile plan. To obtain the necessary degree of accuracy in turning the angles, each transitman turned from his foresight on the movable disk which was set to take care of the odd minutes and seconds of the arc. The vernier was not used. This method gave the location within a probable error of 1 in. The co-operation between the instrument party and the pile-driver crew was so good that a number of times 20 piles were driven in 2 hours under these conditions. Because of the swift current it was necessary then to stop and cap the driven piles in order to prevent their being broken off by excessive and continued vibration.

The main construction equipment of this work consisted of a heavy floating pile-driver, equipped with 90-ft. leads and using a 13 200-lb. double-acting hammer. The outstanding features of this machine was that, in addition to the pile-driving equipment, it carried a revolving derrick with a 75-ft. boom and had an unusual number of engine drums and winch-heads. The machine carried four 2 700-lb. anchors, each of which was handled on wire rope and operated by steam. In addition, it could carry two or more bow and breast lines; it also carried two heavy jet pumps, delivering water at 250-lb. pressure, through 2½-in. jet pipes.

Because of the need of pulling the falsework piles after the pier was completed, the machine was not rigged for batter piles, which were driven by out-hauling the hammer instead of battering the leads. It may be of interest to note that the piles in question ranged in length from 100 to 125 ft., with a 24 to 26-in. butt, and a minimum tip of 10 in.

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[3] Siems-Helmers, St. Paul, Minn.

The second important item was a 2-yd. floating concrete plant with a 104-ft. tower. It was equipped with its own "whirley" derrick for feeding the bins, as well as a bow derrick for handling spouts and tremie pipe. This machine handled tremies as long as 160 ft. for under-water concrete. Its normal capacity was 60 cu. yd. of concrete per hour. It was equipped with weighing devices to permit rapid change in aggregate proportions. It also carried extra engines to handle four anchors on wire-rope lines.

In addition to the foregoing there were two heavy floating derricks, five stiff-leg derricks erected on the falsework of each main pier, five tugs, a locomotive crane, a pump barge, a complete machine shop, and a number of deck lighters. The cement was handled in two heavy barges, housed, and provided with an inclined belt conveyor, power-driven, which delivered the cement on the platform back of the mixer on the concrete plant barge.

Practically all the machinery used was operated by steam, with oil as fuel. The tugs were operated by Diesel engines. All the plant used on this job was the property of the contractor.

Pile-Driving.—At each of the eight main piers, as soon as the first falsework piles were driven, a carefully detailed report of its behavior was wired to the contractor's consulting engineer, who estimated the depth to which the steel shell would sink and determined whether or not the mud should be removed from the inside of the shell before placing the sand filling. These studies and recommendations proved wonderfully accurate in forecasting the behavior of each caisson.

One of the important items in handling this job was the behavior of the mud on the bottom of Suisun Bay. It has been demonstrated that ordinary river silt gives sufficient bearing to be entirely satisfactory for the method used. Only a slight settlement occurred at Piers Nos. 13 to 18, where ordinary river silt was encountered.

When falsework piles were driven at Pier No. 12 a very soft bottom was found. The 100 to 110-ft. pile sank 43 ft. under the weight of the hammer and pile, and moved 5 ft. under the last ten blows. Both shell and caisson showed some movement under these conditions, and when the dredging in the caisson had gone through the sand-filling and had relieved the mud under the cylinder of sand, both the steel shell and caisson listed so much that, at one time, the caisson was 18 in. out of plumb and the shell nearly 8 ft. At certain times the dredging would be 5 ft. below the bottom of the dredging wells on the high side (which was the maximum permitted), and as much as 12 ft. above the bottom of the cutting-edge of the low side. The caisson was landed within 6 in. of the correct theoretical position.

The softest ground of all was at Pier No. 19. The material was described on the original drawing as "blue mud with vegetation" and the falsework piles used were from 120 to 126 ft. long, extending practically to the rock. From 100 to 110 ft. of shell was used in the sand island for this pier. The weight of the sand island pushed the soft material from under it, moving a part of the falsework upward with the flow. Instead of settling uniformly at all times the shell and its sand-filling settled spasmodically, sinking 3 or 4 ft.

on one side and then leveling itself in the next few hours. On April 22, 1930, this shell was 7 ft. out of level.

An attempt to cure this by loading with sand and jetting on the high side was practically unsuccessful, but by dredging for 24 hours on the high side, the shell was brought to within 1 ft. of level, when the dredging was stopped. Additional sand was then added to get it still nearer level. The reason for this procedure was to obtain most, if not all, of the settlement before work on the caisson or pier proper was begun.

An interesting detail of the method used was that the maximum weight is secured at the start; that is, when the sand-fill was completed and 25 ft. of caisson added. From that point the weight of the sand removed and the water displaced by the concrete outbalanced progressively the increased weight of the concrete caission.

Under this method of sinking, the sand between the steel shell and the caisson can always be removed, thus relieving skin friction and giving the greatest weight desired to permit rapid sinking. Under normal conditions a caisson can be sunk at the rate of 10 ft. in a 4-day cycle, and, at times, this was reduced to 3 days.

E. J. SCHNEIDER,[4] M. AM. SOC. C. E. (by letter).—As long ago as 1910, the writer saw designs of a proposed structure across Suisun Bay. Nothing really developed until about 1928, when the psychological moment seemed to appear, and it was apparent that the two car-ferries would soon have to be replaced.

The Engineering Department of the Southern Pacific Company was ordered to make an estimate showing that it would be possible to build the bridge. For preliminary purposes, more than twelve estimates were made of steel for various combinations. Single-track, double-track, and gauntlet-track were considered, as well as high-level and low-level. The "low-level" requirement was that the under-clearance would be 20 ft. above mean higher high water, and the "high-level," 70 ft. Studies were made to determine whether a highway should be built in connection with the bridge, and estimates were prepared with and without the highway, as well as for the construction of the highway at some future time. Furthermore, carbon, heat-treated, and silicon steel were considered in connection with continuous, cantilever, and simple spans. This gives an idea of the various combinations under consideration.

The final choice was for a typical span consisting of a through Warren truss designed to carry a double-track load equivalent to E-90, followed and preceded by a uniform load of 7.5 kips per lin. ft. The total weight of the steel was 42 000 000 lb., of which, 25 000 000 lb. was silicon steel used for the main built-up truss members, floor-beams, and stringers; 4 000 000 lb. of heat-treated eye-bars for the bottom chord of the fixed truss spans; and the remaining 13 000 000 lb. was ordinary carbon steel.

Although it is customary to test to destruction, one or two eye-bars from every heat, the question was asked, "How do we know that the ones that are not tested are all right?" This was given much consideration because of the size of the bridge and the amount of money involved, and it was finally decided that each bar should be proof tested. As a result, the 1 807 bars were tested in the full-sized testing machine, at Ambridge, Pa., and every bar came well within the required limit. This confirms the opinion of the engineers of the fabricators who have always maintained that the eye-bar is one of the most dependable sections in a structure.

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[4] Contr. Mgr., Bridge and Structural Dept., Columbia Steel Co., San Francisco, Calif.

Mr. Kirkbride has explained why this particular location was chosen. The type of superstructure was selected because of economy of design, ability to resist earthquake shock, simplicity of repair, and simplicity of erection.

When the preliminary studies were made, it was first thought that a span of approximately 408 ft. would be an economical length; but the economical curve was about horizontal between 400 and 500 ft., and the question was settled when the War Department ruled that the distance between center to center of piers should be 530 ft. This decision was not reached, however, until the Army engineers had made several tests of long tows in the vicinity of the bridge where swift tides and cross-currents frequently occur. Their experiments indicated that a longer span was necessary to provide ample clearance and to prevent long tows from being "side-swiped." The War Department ruled on a 70-ft. under-clearance from mean higher high water for all except the lift span. This means that all river craft can pass under the bridge and only sea-going vessels will be required to use the Government channel where a lift span is provided with 300-ft. horizontal clearance and 125-ft. vertical clearance above mean higher high water.

The difficulties encountered in building the foundation due to the strong current and deep water and great depth of solid rock have been described in the paper. If these factors could have been avoided, the erection would have been much simplified.

All the through spans have the same panel length. The deck span, which Mr. Kirkbride has mentioned as doing so much work, has the same panel length, except that the two end panels were fore-shortened so they would go between the piers instead of resting on them. Reference was also made to the shelf or offset at the top of the caisson proper which was at Elevation —20 below mean lower low water. This is one of the important details required in connection with the erection of this falsework span as this shelf supported a temporary steel bent on which the deck falsework span rested. The erection of the steel began from the south end and in building temporary falsework and turnouts, as much as possible of the material going into the main span was used temporarily. Two 80-ft. girders were used, which later were incorporated in the south approach; and two panels of through 506-ft. deck span were also used temporarily in erection.

The barges for floating operations were those originally designed and built for the erection of the Carquinez Highway Bridge about 7 miles down stream from the Martinez-Benicia Bridge. They were 40 by 130 ft., with 10-ft. depth, and had a capacity of 1 000 tons. When used on the Carquinez Bridge they carried a maximum load of 800 tons. When loaded to

1 000-tons capacity they had a free-board of 1.5 ft., and extra waling strips were provided on the side so that the deck would be kept dry in case of rough weather.

The first span was floated on February 28, 1930. Advantage was taken of the low tide when the barges were placed under the span. Shortly after noon, the floating was completed, and the span was resting on the temporary bents between the piers. The bents supporting this span were of sufficient length so that the span when erected on this falsework would be about 3 ft. above its proper elevation. Hydraulic jacks were used to take care of any adjustment which might be necessary.

When the lift span was swung, the falsework span was returned to the temporary dock where it originally rested and was re-assembled to the remainder of the span. As the lift span was only 300 ft. long, only eight of the twelve panels of this deck span could be used.

It was not quite so easy to land the deck span in its final position as might be expected because the north bank is not at right angles to the track. The rock-fill embankment is at an angle of about 45° and, therefore, it was not possible to warp the north barge alongside the pier as in deep water. The plan adopted was to warp the span in on a skew so that one shoe could be placed in its permanent position. This shoe rested on a turn-table center and the span revolved about this point. A falsework platform with a system of rollers was provided to accommodate the other pedestal. As the weight was transferred to this falsework platform, the inner barge was removed and the span swung into place as the outer barge was warped into its proper position.

M. F. Clements,[5] M. Am. Soc. C. E. (by letter).—In Conclusion (2) of his paper, Mr. Kirkbride lists the artificial "sand island" as one of the unusual features worthy of special notice.

The use of islands for the lowering of concrete or timber caissons is not new. The first use that has come to the writer's attention was a record of construction of a bridge foundation at Pittsburgh, Pa., in the Eighties. The Romans used mounds of rock or artificial islands on which bridge piers were built. They also used timber caissons, and it is reasonable to assume that they used the timber caisson and artificial islands in combination. The use of the rigid steel shell to enclose sand to form artificial islands is new and was suggested by the writer to the contractor, and his proposal to the Railroad Company for the piers at the Martinez-Benicia Bridge was submitted as a substitute for floating creosoted timber caissons.

The plans submitted to contractors in the invitation for bids were examined and the difficulties of building a floating caisson with a high tide, a fast current, and a very soft underlying material, were noted. The high tide and fast current would naturally prevent the seating of a floating caisson, as there would be a continual washing away of underlying material. This wash would then increase the area of floating caisson exposed to current; it

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[5] Bridge Engr., N. P. Ry., St. Paul, Minn.

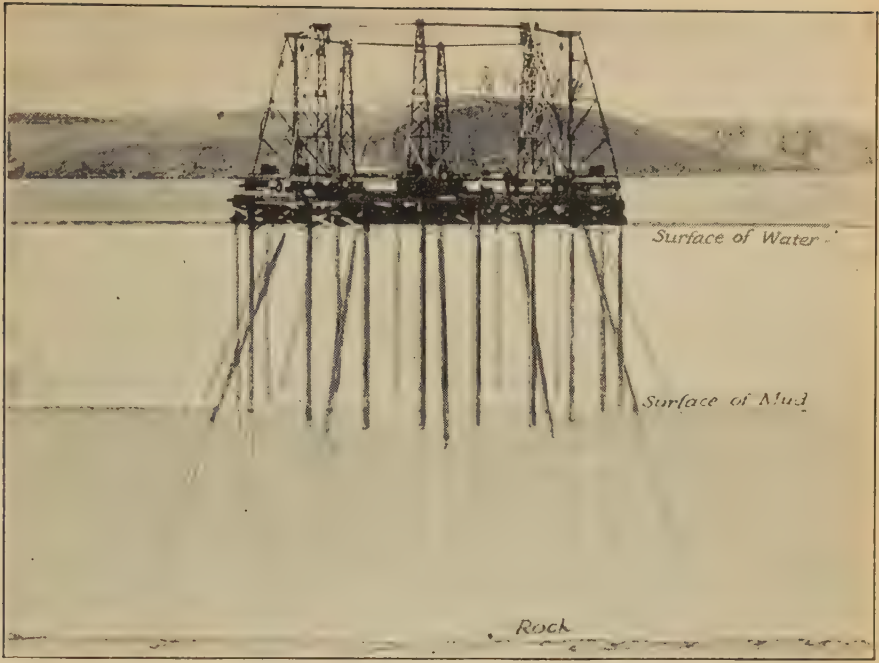
Surface of Water
Surface of Mud
Rock

FIG. 16.—FALSEWORK FOR PLACING STEEL SHELL FOR ARTIFICIAL SAND ISLAND.

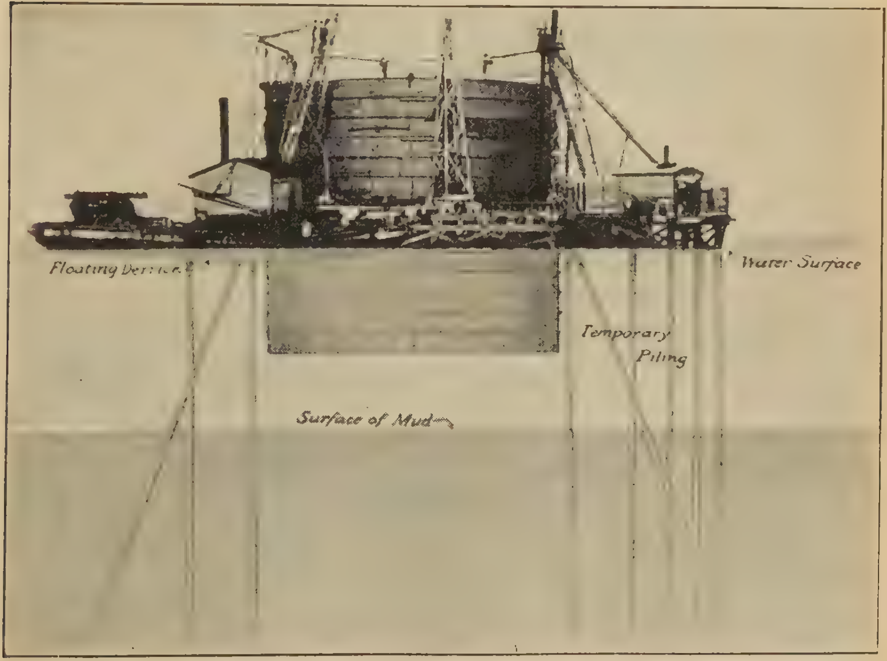
Floating Derrick
Water Surface
Temporary Piling
Surface of Mud

FIG. 17.—ASSEMBLY OF STEEL SHELL FOR ARTIFICIAL SAND ISLAND.

would require more anchorage; and would make it more difficult to locate the caisson accurately. To overcome the foregoing natural conditions the writer recommended the artificial island method as being most suitable for doing the work.

It came to be known on the work as the "sand island" method. Among the advantages in its use, the writer lists a number, as follows:

(1) The element of danger that enters into the handling of large, expensive floating caissons is taken by the lighter, less expensive, preliminary shell, thereby reducing the risk in handling deep-water, deep-foundation construction work. The writer considers the safety element of the use of the "sand island" method to be the most important, and it was the principal reason for recommending its use on the Martinez-Benicia Bridge.

(2) The construction of an all concrete caisson entirely free from the use of timber permits actual contact of the concrete caisson walls and the concrete placed in dredging wells. In salt water infected by teredo or other marine borers, a wooden floating caisson must be protected by an outer layer of creosoted material, which adds to the cost of that type of construction.

(3) Concrete was poured above water where the mixture could be controlled and the deposit made in the dry. In comparison with the floating caisson, this is a distinct advantage in that a floating caisson cannot be made water-tight; therefore, tremie concrete had to be poured, and it would be impossible to obtain 3 000-lb. concrete, such as cited by Mr. Kirkbride, on account of the presence of water in the wall form of the caisson.

(4) With the "sand island" method the bottom part of the caisson is substantially supported by the sand, and high strength reinforced concrete can be made. The structural steel cutting-edge is merely a metal covering which is comparatively inexpensive; whereas, with the floating caisson, the cutting-edge is a rigid frame supporting superimposed water-tight compartments in which concrete is placed. The cutting-edge of a floating caisson is of necessity of greater weight and cost than one used on a caisson constructed on an artificial "sand island."

(5) The structure was spotted accurately over its final location at the beginning of the work.

(6) The caisson was controlled during the sinking process for the entire depth. The sand acting as an equalizer of frictional value for the different classes of material, penetrated below the natural ground line. At this bridge there was no sudden sinking as the caisson entered the mud below the sand in the shell.

(7) The caisson could be controlled in landing on the bottom. At the location of the bridge there was a fluctuation of tide amounting to 10 ft. and a velocity of current of 5.8 miles per hour in either direction, due to ebb and flow tide.

(8) The "islands" weigh the same as the caissons which pass through them and the action of high water, winds, waves, and tides has no greater effect than they do on the permanent piers.

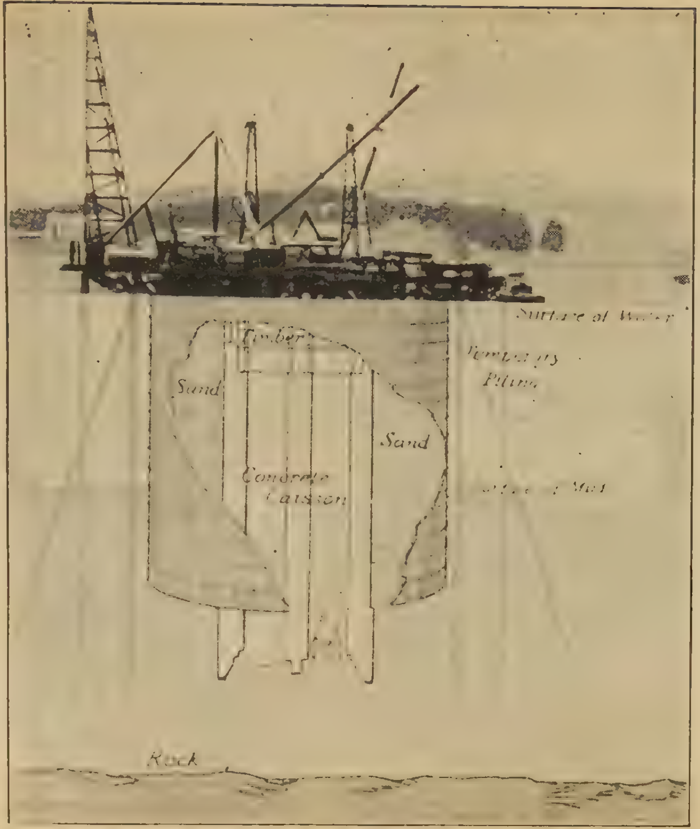
Surface of Water
Timber
Temporary Piling
Sand
Sand
Concrete Caisson
Rock

FIG. 18.—SINKING CONCRETE CAISSON THROUGH STEEL SHELL AND ARTIFICIAL SAND ISLAND.

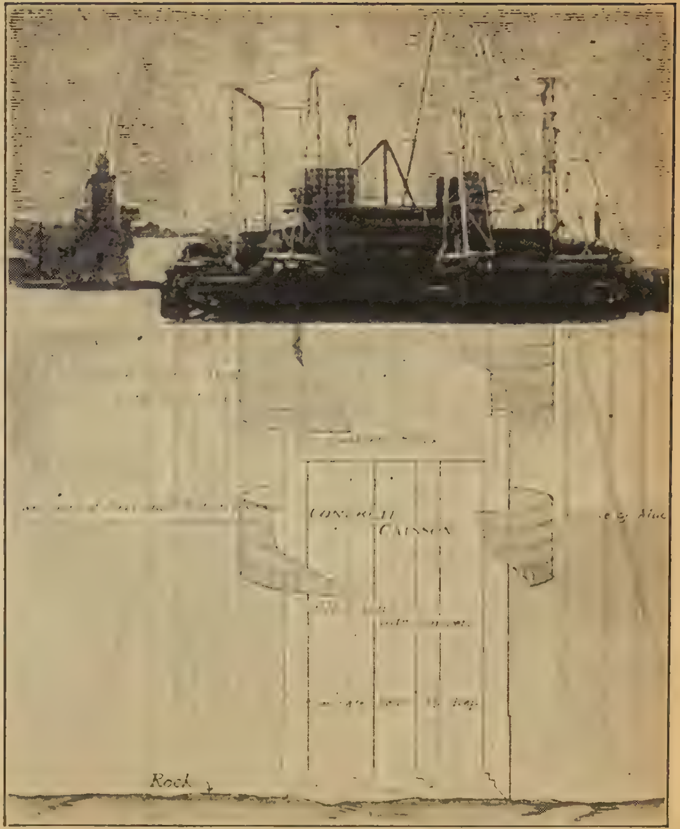
CONCRETE CAISSON
Rock

FIG. 19.—REMOVING STEEL SHELL FROM ARTIFICIAL SAND ISLAND.

(9) The method results in a marked saving in cost that varies in amount by the extent to which shells and falsework can be re-used. The actual costs resulting from the construction of "sand islands" were exceedingly close to the estimate.

(10) Speed of construction by the "sand island" method is the result of a better co-ordination of the work. The speed of sinking the caisson depends upon the speed of its construction and this can be handled with greater speed on a rigid rather than a floating structure. The Martinez-Benicia Bridge was actually completed before the anticipated date.

(11) The method reduces the amount of floating equipment required and can be applied with a minimum amount of fixed equipment.

(12) Steel sheet-piles have been used for enclosing sand in shallow water, but are not of sufficient strength for the greater depths, and had their use been attempted at the Martinez-Benicia Bridge, it probably would have resulted in the loss of two islands and caissons.

(13) The "sand island" method eliminates the necessity of constructing launching ways and providing anchors and anchor barges, which are used in the floating type of caisson.

As a matter of information, it is well to call attention to the predicted movement of islands and caissons during sinking. This was done by analyzing the data obtained on the pile-driving during the construction of the falsework. The shells were assembled on I-beam supports and lowered into the mud. As a matter of fact, the shells were lowered to an elevation 5 ft. above the mud line and then, with all lines released, it was permitted to drop 5 ft. to force a penetration into the mud. The analysis of pile-driving for the purpose of determining the depth to which the steel shells would sink into the mud, was made with surprisingly accurate results. Some difficulty was experienced at the last deep pier (No. 19, Fig. 2), and after the pier had been landed on bed-rock, it was evident, in the light of experience, that all the action of the caisson could have been predicted and all delay avoided.

Another lesson was learned from estimating the possible sinking of the shell into the mud as the result of analyzing the action of the test pile. The value of the pile to support load is both in terms of bearing on the tip and friction on the sides. A pile of known size and approximate weight was lowered until its tip touched the mud. It was then released and the penetration from its own weight noted. A hammer weighing approximately 13 kips was placed on the pile and its further penetration noted. The bearing and friction as determined by these tests were applied to the bearing area and sides of the steel shell and the final depth to which the shell would be sunk, was determined. In most cases, the depth was predicted to within 1 ft. However, that is not so important as the fact that the prediction was made for a shell, the bearing area of which was one hundred and fifty times the bearing area of the pile.

These computations were based on ultimate values. In other words, the ultimate value of the mud or earth in bearing for the larger area was determined by the smaller area. The writer believes that tests made on soils on small areas will give fairly accurate bearing values for larger areas. This has been a disputed point, but the results of the tests of pile-driving at the Martinez-Benicia Bridge tend to prove the statement.

As a result of the experience at this bridge, it is the writer's opinion that deep piers can be placed by the "sand island" method in tidal waters under adverse conditions of high winds and waves, using circular shells 400 ft. in diameter in water 150 ft. deep.

It may appear in first analysis that the cost of all large sizes is excessive, but the safety factor outweighs all disadvantages. It can be used where the abnormal conditions, due to wind and waves, make a floating caisson dangerous.

Under the heading, "Construction," the author had described the steps involved in the artificial sand island method. Fig. 16 is a view showing piles driven to support the octagonal platform. It also shows the eight gallows frames mentioned in the paper. In Fig. 17, the steel shell is being erected and lowered. A view of the third step, showing the concrete caisson being driven through the sand island to solid rock, is given in Fig. 18 and, finally, Fig. 19 demonstrates the final step of removing the steel shell.

Jacob Feld,[6] Assoc. M. Am. Soc. C. E. (by letter).—The method of sinking caissons for the deep-water piers is one of the most noteworthy items in the design and construction of this bridge. Although it is not a new idea, the development of the technique and the success of the operation should result in the standardization of open-caisson construction where depths of water are not excessive.

The use of an artificial sand island for each pier eliminates the dangers from tide and current in making difficult the sinking of a caisson and also provides an easy method for the construction of the cutting-edge in true location.

The two main difficulties in caisson construction have been floating the pre-cast or lowest section (usually built on shore) and the danger from tipping during sinking. The use of a sand island eliminates both dangers. In addition, careful explorations can be made of rock contours beneath the proposed caisson and the cutting-edge can be built to suit such conditions, which practically guarantees a perfect fit when the caisson reaches rock.

Of course, when depths of water are excessive the cost of an artificial sand island may be prohibitive. In addition, the skin friction induced by the sand that has been placed artificially, increases the cost of sinking the caisson. This method might be revised in deep-water conditions by providing a steel shell enclosure driven to mud, and using such shell as the foundation for a working platform upon which the cutting-edge can be set, the lower part can be built, and the entire caisson floated therein. As a matter of record such a procedure has been used for the Grey Strait Bridge, at Brisbane, Queensland, Australia, described subsequently.

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[6] Cons. Engr., New York, N. Y.

As far as the geology of the ground is concerned a comparison of the sub-surface investigations reported by Mr. Kirkbride, with the result of similar investigations made for the Carquinez Strait Bridge, shows almost identical conditions.[7]

The nature of the rock found at that location was the same as at the Martinez-Benicia Bridge. In addition, it was found that at the Carquinez Strait Bridge layers of sandstone and shale were often separated by beds of softer material, such as clay and in some cases gravel. It was also found that the deeper water was not above the old filled-in channel. As a matter of fact, Pier No. 4 of the Carquinez Bridge, which is above the original deep channel, is founded on piles because sound rock was not found at a depth of 200 ft. below mean high water.

Within the ten years, 1922-1932, there have been some noteworthy examples of deep caisson construction, including at least ten examples of foundation construction in which caissons were sunk more than 100 ft. below either the ground level or the water level. A short description of each of these examples should be of interest because they show the rapid development of this type of construction.

(1) In 1923, for the Rigolets Bridge, of the Louisville and Nashville Railroad Company, about 30 miles east of New Orleans, La., five steel shell caissons, 34 ft. in external diameter, with single dredging wells 16 ft. in diameter, were sunk to total depths of 80 to 113 ft. Under 45 ft. of water there was a considerable number of layers of soft black mud overlying stiff yellow clay, fine sand, and some gravel. These caissons were floated into a ring of steel sheet-piling partly driven before the caissons arrived, and the remainder of the ring was completed after the caisson had been located. The caissons were sunk entirely by open dredging methods and by the addition of concrete between the two steel rings that form them.

(2) In 1923, at Lexington, Mo., two steel cylinders, 14 ft. in diameter, were sunk to a total depth of 110 ft. Under 13 ft. of water there were various layers of sand, blue clay, gravel, and boulders. The caissons rested on soapstone. Most of the excavation was done under air, a maximum pressure of 50 lb. being used.

(3) In 1926, for the Carquinez Strait Bridge, in San Francisco Bay, caissons with a dimension of 40 by 40 ft., and 112 ft. in length, were floated into position and sunk by open dredging methods. They were built of timber with a steel covering for the cutting-edges, but with concrete posts so that, in the event the timber surface was removed the timber cross-bracing would not be exposed to the action of salt water and wood borers.

Before floating the caissons into position a guide-frame consisting of six steel spuds, 120 ft. long, was built up and anchored by four anchor barges. The conditions at the pier were 80 to 90 ft. of water overlying various layers of mud, coarse sand, gravel, and thin layers of clay, as well as shale and sandstone. The caissons were sunk to a maximum depth of 135 ft. below water level. It was noted that the friction-reducing jets placed on the exterior faces of the caissons were not of much value, whereas the cutting jets placed on the interior faces of the caissons were useful in assisting the sinking.

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[7] "Design of Substructure of Carquinez Strait Bridge, California," by Jacob Feld, Assoc. M. Am. Soc. C. E., *Proceedings*, Brooklyn Engrs. Club, Vol. 23, April, 1925, pp. 3-34.

(4) In 1927, for the Kennebec Bridge, at Bath, Me., three of the seven caissons were sunk more than 100 ft. Two of these were 22 by 40 ft. and the third was 25 by 54 ft. The caissons were of wood, built 20 ft. high, floated, and held in position by six Chinese anchors. The excavated material was chiefly plastic blue clay and coarse sand. The caissons rest on granite rock. The excavation was done chiefly under air.

(5) In 1927, for the Umgeni River Bridge, in South Africa, six caissons, 12 ft. in diameter, were sunk to depths of 60 to 156 ft. Because of the extreme low water, sections 20 ft. in height were built in place and sunk through coarse sand and clay by open grab-bucket excavation. The caissons were built with 2-ft. walls and steel cutting-edges and sealed by tremie concrete.

(6) In 1928, for the Detroit River Suspension Bridge, between Detroit, Mich., and Windsor, Ont., Canada, two caissons, 38 ft. in diameter, were located in water varying from 8 to 25 ft. in depth. At these locations, artificial islands were built by pumping sand to above water level, and the caissons were started on top of this fill. In addition to these caissons, there were two other caissons 38 ft. in diameter and two anchorage caissons each 100 by 225 ft. These caissons were sunk by open dredging through mud and sand and then blue clay about 75 ft. in thickness, and varying layers of quicksand from 20 to 62 ft. in thickness. When the caissons reached hardpan (a layer approximately 10 ft. thick, overlying limestone rock) that material was excavated under air, and the caissons were sealed under air.

(7) In 1928, two caissons were sunk for the Mid-Hudson Bridge, at Poughkeepsie, N. Y. Each caisson was 60 by 136 ft., with a steel cutting-edge and a steel shell 20 ft. deep. A temporary wooden bottom had to be installed for floating, and the caisson was held in position by eight 15-ton anchors. The conditions were 60 ft. of water and 18 to 35 ft. of mud overlying stiff clay, sand, and gravel, the east caisson going down to Elevation − 114 and the west one to Elevation − 134. All material was excavated in the open, and considerable difficulty was encountered as has been explained in recent engineering literature.[8]

(8) In 1929, for the Federal Barge Line Terminal, on the Mississippi River, several circular caissons, 10 and 12 ft. in diameter, with 2 ft. 0-in. and 2 ft. 3-in. walls, using steel cutting-edges and steel shell forms, were sunk through 80 ft. of water and 40 ft. of sand with some blue clay. In these long and narrow caissons it was found that a balanced pressure must be maintained to prevent tipping. Sinking was performed by clam-shell excavation with the assistance of jets on the outside of the caissons.

(9) In 1929, for the Grey Strait Bridge, in Brisbane, Queensland, Australia, four caissons, 28 ft. in diameter, were sunk from 96 to 115 ft. through artificial sand-filled islands consisting of steel sheet-piling driven on a diameter of 30 ft. The first section of the caisson was built on this artificial

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[8] *Engineering News-Record,* February 12, 1931, pp. 275–281.

sand island and was sunk by open dredging until rock was reached. The caisson was then sunk an additional 5 ft. into rock under air, using a maximum pressure of 50 lb.

(10) The latest example is that of the sixteen caissons sunk inside of 81-ft steel cylinders described in this paper, and it will be seen how the difficulties of the previous constructions have been eliminated by the methods described so ably by the author.

W. H. Kirkbride,[9] M. Am. Soc. C. E. (by letter).—The writer is indebted to those who have discussed his paper, particularly for the additional information presented.

Mr. Helmers calls attention to the use of triangulation in measurements and layout. The site of the bridge is such that direct measurements were difficult, because of the depth of water, the strong currents, and large volume of water traffic. He also mentions some of the difficulties encountered in sinking the cylinders to retain the sand of the sand islands. It must not be assumed that sinking caissons by this method is free from difficulties, but the ingenuity shown in straightening the cylinders and caissons after they had settled out of level, and in maintaining the caissons in a vertical position, resulted in landing them quite close to their true position and well within the tolerances allowed.

Mr. Clements gives additional information and points out the advantages of the use of the "sand island" method, in which the writer fully concurs. He directs attention to the fact that while the use of artificial islands is not new, the use of the rigid steel shell to enclose sand to form the artificial islands, is new. The writer recognizes this fact in his Conclusion (2), in which he referred to "artificial islands of sand within steel shells." Credit for the use of this method is due to Mr. Clements, who, as Consulting Engineer for the Contractor, was responsible for the suggestion that the method be used, and for working out the details of construction.

Mr. Schneider calls attention to the "proof-testing" of all the heat-treated eye-bars used in the bridge. When the specifications for the superstructure were under discussion, heat treatment of eye-bars was rather new and to be certain that all bars were of equal quality and conformed to the requirements, arrangements were made to "proof-test" each bar. The eye-bars before proof-testing had their pin-holes bored to a diameter sufficiently smaller than their final diameter so that after testing they could be re-bored to exact length and pin-hole diameter, thus removing any permanent set. The proof-testing consisted of applying to the bars a load of 36 000 lb per in. of nominal cross-section.

The re-boring of the pin-holes was supposed to take care of only the slight distortion that might occur. Bars with any permanent set in the body of the bar were to be rejected. It is gratifying to know that all the bars proof-tested were accepted; they were all tested to destruction and complied fully with the specifications.

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[9] Chf. Engr., Northwestern Pac. R. R.; Engr., M. of W. and Structures, S. P. Co., Pacific Lines, San Francisco, Calif.

Mr. Feld gives valuable information in listing examples of deep caisson construction, and calling attention to the progressive development during the past ten years.

Mention was made in the paper of earthquake faults near the bridge and of efforts to design the superstructure and foundation to resist earthquake vibration. Since the completion of the bridge, the United States Coast and Geodetic Survey has placed a strong-motion seismograph on top of one of the piers with the expectation that the record obtained may be of assistance to engineers in developing more accurate principles as to the design of bridge piers to resist earthquakes. Similar machines have been installed in various places in California. Some difficulty has been experienced in designing a starting device that will be sensitive to slight earthquake shocks, and at the same time, not respond to other vibrations, such as may be produced by the starting and stopping of trains on the bridge, but it is believed that this trouble will soon be eliminated.

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PRE-QUALIFICATION OF CONTRACTORS[1]

By C. J. Tilden,[2] M. Am. Soc. C. E.

With Discussion by Messrs. Edward W. Bush, A. B. Edwards, Samuel T. Carpenter, Leon F. Peck, C. L. Hall, A. R. Losh, E. G. Walker, and C. J. Tilden

Synopsis

A definition of pre-qualification of contractors is offered for discussion in this paper. This is supported by a brief historical comment, a statement as to legal aspects, and opposing viewpoints. Advantages and disadvantages are presented concisely, and the conclusion is reached that some form of pre-qualification is desirable from every point of view. Conditions must not be made so rigid, however, that new contracting firms would be prevented from entering the field.

Definition of Pre-Qualification

The principle of pre-qualification may be defined as "the determination of responsibility before plans and proposals are issued for bidding." This determination is based on: (1) The contractor's previous experience; (2) the kind and condition of his equipment; (3) his record on previous work of a similar character; (4) his financial condition; and (5) his character and general reputation. A more extended definition was incorporated in the California Law of 1929 (Chapter 644):

> "The Department of Public Works may, within its discretion before furnishing any persons proposing to bid on duly advertised public work, with plans and specifications for the proposed public work, require from any such person answers to questions contained in a standard form of questionnaire and financial statement, including a complete statement of the person's financial ability and experience in performing public work.
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> "Whenever the Department of Public Works is not satisfied with the sufficiency of the answers contained in such questionnaire and financial state-

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[1] Presented at the meeting of the Highway Division, New York, N. Y., January 22, 1931.

[2] Strathcona Prof. of Eng. Mechanics, Yale Univ., New Haven, Conn.

ment it may refuse to furnish such person with plans and specifications on any such duly advertised public work, and the bid of any person to whom plans and specifications have not been issued must be disregarded."

In a bulletin issued by the Bureau of Municipal Research, Philadelphia, Pa., in April, 1929, the following comparison was given:

"Prequalification is but an extension of the principle applied to the professions, where the possession of theory and practice must be demonstrated by the applicant and attested by diploma and State board; and in the trades, where those who would be masters must serve apprenticeships."

It is seen from the foregoing that the principle of pre-qualification is not new. It has been used to advantage for many years in private work, where the owner receives bids only from a restricted and especially invited list of contractors. The attempt to incorporate into law the principles defined herein is comparatively new, however, and has received its chief stimulus from the rapid expansion of highway building in the last few years.

History of Pre-Qualification

It has always been a generally accepted principle that municipal and other authorities had the right to withhold the bidder's sheet from a contractor to whom the award would not be made, no matter how low his bid. Similarly, public officials have always had the right to require some kind of statement of the prospective bidder's qualifications. This is usually in the form of an answer to a more or less carefully prepared questionnaire. It was incorporated into the State Laws of Wisconsin in 1925, Iowa in 1927, and California in 1929. It is required or enforced either under existing specific statute or by administrative procedure in a number of other States, and, in addition, it is used in many municipalities. Proposed legislation on the subject failed, however, in Oregon in 1929, the chief cause being a provision in the bill to the effect that if a prospective bidder takes an appeal (from disqualification), no bids shall be opened or made public until this appeal has been determined. Such a provision might evidently cause serious delay. In 1929, also, the Governor of Pennsylvania vetoed proposed pre-qualification legislation on the ground that it placed too much discretionary power in one city official, without opportunity for redress on the part of the disqualified bidder.

The recent movement in favor of pre-qualification seems to have been started about 1924 by the contractors themselves, at a meeting of the Committee of Associated General Contractors of America and the American Association of State Highway Officials. It has had the enthusiastic support of the better class of contractors ever since that time.

Legal Aspects of Pre-Qualification

Apparently, the Courts have not yet threshed out thoroughly the legal principle of pre-qualification. A city ordinance providing for pre-qualification of bidders on municipal work in Philadelphia was the basis of a suit brought by a bidder. The Court decided that the ordinance was illegal, but on appeal the judgment of the Lower Court was set aside. The case was reviewed by the Supreme Court, which declared the ordinance unconstitutional, but made several suggestions as to changes that might be made, which would keep the intent of the statute and yet be within the Constitution. One of these was the desirability of a Board of Review, composed of persons in no way connected with the department that was advertising for bids, in order to assure the bidders unbiased consideration.

This is naturally not satisfactory to city engineers. It means that they must give up a duty and responsibility that they are loathe to relinquish. Furthermore, it does not appear to be in the public interest, for engineers within the department are certainly the best fitted to handle that phase of the work. A new pre-qualification ordinance was passed by the City Council of Philadelphia and became effective September 5, 1930. This was written to include the provisions held by the Supreme Court to be essential.

The Contractor's Standpoint

From the standpoint of the general contractor, pre-qualification is decidedly advantageous. Responsible firms generally are hoping that the public will differentiate between classes of contractors, giving due weight to reliability, financial responsibility, and character, rather than making cheap price the controlling factor. Frequently, only two qualifications are required, the first being the lowest bid, and the second a satisfactory surety bond. This often militates against the responsible contractor who enters into his agreement expecting to abide by the plans and specifications and who has had experience in the technique of his work.

The surety bond is by no means complete insurance of satisfactory performance. Pre-qualification, as defined herein, will benefit both the bidder and the public. The Associated General Contractors of America has expressed itself as strongly in favor of the principle and has established a Bureau of Contract Information to co-operate with public officials and others who are investigating performance records, reputation, and general responsibility of contractors throughout the country.

Failure of Contractors

The public generally loses thousands of dollars annually in failures on contracts that have been let on the basis of the lowest bid to incompetent contractors. A large surety company has classified the causes of its losses, as follows:

	Per cent.
Inadequate financial ability	50
Incompetency	25
Dishonesty	15
Miscellaneous items, such as weather, bad luck, etc.	10

A study undertaken by the Bureau of Public Roads, United States Department of Agriculture, to determine the reason for unsatisfactory progress on Federal Aid highway projects, finds poor management the most important factor (52.7% of the cases studied), with weather (26.4%) second on the list. Inadequate financial ability, construction difficulties, and minor causes are also mentioned.

Disadvantages of Pre-Qualification

A number of objections to the principle of pre-qualification have been stated. It is claimed that competition is unduly restricted by the limitation put on the number of bidders and especially that small firms struggling to get a satisfactory hold in the industry are barred from bidding. It is claimed further that the usual requirement of a certified check is in itself sufficient evidence of competency. Objection has been made to the questionnaire that must be answered as a preliminary to putting the contractor on the preferred list, as an unwarranted intrusion into private affairs.

Public officials who are charged with the duty and responsibility of letting the work are naturally biased in favor of more competent firms of whose previous work they know. This, of course, is a serious disadvantage to the new or little known bidder. If the principle of pre-qualification is too rigidly followed, it will prevent new men from coming into the field, and as existing firms drop out, there will be no others to fill their places.

Advantages of Pre-Qualification

One of the most desirable results of pre-qualification, from the business standpoint, seems to be the increased confidence felt by manufacturers of contractors' equipment, bankers, and the surety companies. From the engineer's point of view the advantages are obvious, because he can feel much greater assurance that the plans and specifications will be followed properly. The contractors have expressed the firm belief that the principle is of benefit both to the bidder and to the public, and have supported this belief with practical co-operative efforts which tend to simplify and facilitate putting the principle into practice where it has been established.

With sworn statements at hand as to the contractor's experience, equipment, finances, and the number of contracts he has at the time, engineers can feel more confidence in the prompt and satisfactory execution of the work, and the banks and surety companies can feel greater confidence in the financial and economic phases of the question. As a protective measure to the legitimate and responsible contractor it is an important advantage.

Conclusions

There seems to be little doubt or controversy among intelligent and responsible individuals, either engineers or contractors, as to the wisdom of some form of pre-qualification. With this practical unanimity, however, there is the feeling that conditions must not be made too rigid or the statutes too exacting. Although it is highly desirable to protect the interests of the public and also those of the high-grade contractor, there should be enough leeway in the exercise of judgment of governmental officials to permit new contracting firms to enter the field.

Some form of questionnaire that will exhibit fully and clearly all the facts with regard to the bidder's financial responsibility, general character, and probable ability to complete the work satisfactorily, is highly desirable.

DISCUSSION

Edward W. Bush,[3] M. Am. Soc. C. E. (by letter).—This paper is timely and it is hoped its publication will lead to a general discussion of the subject. Pre-qualification is economic, but just now the procedure needs protection from some of its most ardent friends. Some of them are suggesting methods that are too complicated to be workable; they aim to develop certain facts of little importance; and they omit the development of other facts of prime importance on which the determination of the fitness of the contractor for the award necessarily depends. This question of the determination of the qualification of the bidder to receive the award is exactly parallel to a part of the method followed by any reputable surety company when judging the acceptability of a contract bond case; but an examination of many of the questionnaires indicates that their authors have not recognized this fact and, apparently, no attempt has been made to benefit from the very extended experience of the sureties along the same line. A surety is keenly interested in the fitness of a contractor to perform the contract properly, because if it errs in its judgment on the case it may suffer a loss. All large sureties are in touch with all kinds of construction projects in all parts of the United States and have learned by experience that human nature re-acts in about the same way on all of them. The sureties have a seasoned experience on the methods of determining the qualifications of a contractor, which if called for would assist those about to establish any scheme of pre-qualification.

When a contractor fails he is generally forced to retire from the business or continue with greatly lessened activity. Just why he failed may not be fully recognized by him, because he had never failed before, and he is apt to think that all the "breaks" were against him, etc. One never listens to a technical paper read by a contractor telling why he made a mess of his business. Active and successful contractors therefore are, perhaps, not the best qualified to outline workable pre-qualification schemes because they have not been intimately associated with trouble cases as are the sureties that pay the claims. Engineers, also, are apt to lack any extended experience in the defaults of contracts. If at any locality defaults were of frequent occurrence the premium rates would be increased greatly over the present small charge for the bond protection.

The writer is not in full accord with some points of the paper. The author defines pre-qualification as "the determination of responsibility before plans and proposals are issued for bidding." Any such procedure would necessarily cause a known list of prospective bidders to be made up well in advance of the letting, and this might prevent an outsider from breaking in on any local group that possibly might have matters fixed to its entire satisfaction. The main object is not to make an award to an unqualified bidder, but this is quite removed from a scheme of pre-qualifying all the bidders before giving out the plans. A more workable way is to state in the call for bids that no bid will be received and opened from a bidder who has not qualified himself for the contract. With such a procedure most, if not all, of the prospective bidders will qualify well in advance of the letting, but all of them will know that it is possible for some other bidder to qualify himself just before the opening, and all the benefits of open competition are thus preserved. Many contractors will not know until after they review the plans and visit the site whether or not they wish to bid, but the author's definition outlines a procedure which would tend to keep this class away from the bidding.

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[3] Engr., Aetna Casualty & Surety Co., Hartford, Conn.

It takes time and considerable skill to verify and evaluate properly the technical and financial information submitted by any unknown contractor, and such a check should be made by the owner before making an award to any contractor not well known to be qualified. Sufficient time can be taken after the opening for such an investigation, but to make a complete investigation of all bidders before giving out the plans would be a huge task the time for which is seldom available unless the owner is willing to receive the bids from six to eight weeks after the advertisement appears. Since adequate time is, in general, not available and since, considered as a practical matter, about all the owner can expect to accomplish before the opening is to collect enough data to enable him to form a preliminary opinion that the contractor should be allowed to bid, the owner making a complete check of the bidder's affairs only if he happens to be the low bidder. This is a practical way to work pre-qualification.

If the general information on a contractor has already been filed it is available of course for subsequent lettings; therefore, only a comparatively small amount of additional up-to-date information is needed to develop a rather full understanding of the status of the bidder at future openings. It is desirable to have the pre-qualification scheme as simple as possible both in forms and procedure so as to attract instead of repel the best bidders.

The author presents parts of a California law as "a more extended definition" of pre-qualification. These excerpts show that, at its discretion, the Department of Public Works may call for questionnaires and financial statements before giving out the plans to bidders; and if the Department is not satisfied with the "sufficiency of the answers" it may refuse to furnish the bidder with the plans, etc. Apparently, the Department can neither follow up references nor verify the financial worth of a prospective bidder as it can only read the answers submitted. It is not charged with the duty of making an investigation other than along the narrow lines of the Act. No doubt the Department does not restrict its investigation to the narrow limits just stated, but the Act is not, in itself, a good definition of a workable scheme of pre-qualification. Further, it might be inferred from the wording that a contractor must have had a previous "experience in performing public work" in order to qualify, and this might bar a well qualified contractor if his experience had been gained wholly on work for a railroad company or a public utility. A better definition would be along the line of the following:

> No bid shall be received until the bidder shall have shown before the bids are opened that he has had a comparable experience in the same or similar classes of work (if a corporation, the information shall be submitted on

the experience of the personnel), also has or can obtain sufficient plant and equipment, also is possessed of sufficient financial resources as reasonably to justify the conclusion that the bidder is qualified to perform the proposed construction contract as well as continue any other contracts on hand.

In each case decisions regarding the prospective bidders shall be reached before the bids are opened, and if a prospective bidder is deemed unqualified his bid shall be returned if offered, and it shall not be received and opened.

Such a wording, if incorporated in a law or ordinance, leaves the officials a desirable freedom of action regarding the method of pre-qualification. At times an informal method could be followed. A further investigation before the award is not disbarred because all awarding bodies are now supposed to exercise discretion in the award; the new matter charges them also with the duty of investigating the bidders before the opening.

The author lists five points or elements that determine the principle of pre-qualification. He ignores other points such as the skill and ability of the contractor and the skill and experience of the contractor's organization. The surety company always collects information about these points in connection with a large contract because it wants to know that the organization will continue to function in the event that the contractor personally becomes incapacitated. Another most important element is omitted by the author as well as in the California Act, namely, information about the size, location, kind of work, and approximate percentage completed of all other contracts on hand. Are the other contracts progressing satisfactorily and profitably? A contractor has only one plant and one financial strength with which to perform all his contracts and, in the past, many new contracts have been taken by contractors with the hope that the new work will in some way supply funds needed to bolster up bad conditions already developed on other jobs. No scheme of pre-qualification is workable that neglects to give full consideration to the uncompleted work of the bidder. The sureties always investigate the other work, except in connection with small bonds written for strong contractors about whom rather full information is at hand.

The author's Point (2) indicates that the condition of the plant should be considered. It may be located here and there on other work and its condition would be impossible to determine except at a prohibitive cost. It is desirable to ask the average contractor to submit a list of the large pieces of plant, to state specifically what new plant, if any, will be needed to perform the new job if awarded to him, and to state the cost of such new equipment. The cost of new plant comes from the present working capital or net quick assets of the contractor, and, therefore, the question is always asked by a surety when underwriting a contract bond. The fact that a contractor owns plant of a certain kind is an indication that he has had experience doing the kind of work suited to the plant, and information of this kind is needed when qualifying a contractor. It is possible for a contractor to be well qualified and not own any plant, provided he knows what he should buy and has ample funds with which to purchase. The Hoover Dam was let to a new corporation having nothing but a personnel of marked ability and several million dollars in the banks.

The author presents the history of pre-qualification, its alleged advantages and disadvantages, etc., in a concise and admirable manner. The present trend among engineers and public bodies is toward a more careful selection of contractors before receiving the bids, and toward making investigations before awarding contracts. Reputable contractors favor these practices because they prevent unfair competition. The sureties, also, are in favor of the exercise of more discretion by the owners when placing contracts because no surety ever has the chance to bond an unqualified contractor until after a contract has been awarded to him. The author points out the interest the Associated General Contractors of America has exercised in helping to establish the Bureau of Contract Information, Inc.; but, apparently, in some way he missed the fact in his investigation that from its inception a select group of surety companies also has been active in the affairs of the Bureau. To date this group of sureties has contributed the major proportion of the funds used to maintain the Bureau.

The author closes his admirable paper by stating that some form of questionnaire is highly desirable, which will exhibit fully and clearly all the facts, etc., regarding a contractor. It is comparatively easy to qualify or underwrite a capable and experienced contractor as long as he keeps the scope of his operations well within the amount warranted by his working capital. When he attempts to go beyond his safe limit of volume of work more investigation of his affairs is needed and mostly this is in the line of checking carefully his alleged financial strength. The manifestly unfit are quickly disposed of, but the class between these two is the most numerous and it is also the class that causes, or should cause, the most investigation. Merely requiring answers to certain questions will not in itself establish for this class the fitness or disqualification of the bidder to receive an award, although the answers will probably direct the course of the inquiry. Some of the questionnaire forms are good, but more of them are very poor working tools for an engineer or a public works body. Any of them could be improved and as this important subject is still in a state of flux, it would seem desirable that an organized study should be made to develop the best thought regarding it. Recently, many bills on pre-qualification have been offered to State legislatures. Some of these bills have proposed non-workable schemes advocated by zealous but poorly informed individuals or societies. Many persons desire the benefits of pre-qualification, but do not know a good workable way to obtain them.

A. B. Edwards,[4] M. Am. Soc. C. E. (by letter).—To accept a bid without a demonstration of the aptitude, ability, integrity, and experience of the bidder would be economically foolish. In contrast with this clumsy procedure is the policy of determining the qualifications entering into the make-up of the bidder who can and will co-operate with the engineer to give the owner his money's worth.

The bidder should be able to assume full responsibility for the planning and execution of the work of construction in accordance with the commercial requirements of the owner, as outlined by the engineer. The bidder in whom confidence and responsibility is thus placed, will be developed by pre-qualification.

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[4] Asst. Engr., Div. of Met. Planning, Commonwealth of Massachusetts, Boston, Mass.

One potent aid in the establishment of pre-qualification as a permanent policy may be the equitable handling of candidates for membership in the fraternity of bidders. There are those who have served their apprenticeship without submitting to the enervating influences of expediency, who are adequately trained in sound procedure, and who are worthy of financial backing. Pre-qualification can and should give these canditates "elbow room" by combating the unworthy element.

Samuel T. Carpenter,[5] Jun. Am. Soc. C. E. (by letter).—That some form of pre-qualification or qualification is necessary to determine the responsibility and financial standing of contractors is undeniable. To what extent it can be developed in order to place no restriction on competitive bidding is problematical. For the public's interest the presentation of a surety bond should not be taken as conclusive evidence that the finished product will be satisfactory. Specifications for public work usually carry a clause demanding the contractor to guarantee his work fully from defects for at least a year after completion. In the writer's opinion too much dependence is placed upon this clause to insure a satisfactory product. The engineer deserves a finished product that will be a tribute to his engineering skill, and the legitimate contractor deserves a protection against those who are obviously unable to complete a building program successfully, either financially or technically.

Professor Tilden states five basic criteria for determining the fitness of a contractor to receive plans and proposals for bidding. In addition, it might be well to investigate his organization, personnel, and their experience.

In 1929 the State of Indiana adopted a form of qualification that provides for a questionnaire to be submitted by the contractor with his proposal. Section 1 of the Indiana statute[6] that covers this form of qualification stipulates that when any public work aggregating $5 000, or more, is to be constructed or repaired under contract, it must be let to the best of the lowest bidders. Section 2 provides means of classifying the bidders. Each bidder must submit a sworn statement of his experience, his proposed plan, and the equipment he has available for doing the work. Standard forms for this purpose (General Form 96-*a*) are supplied by the State Board of Accounts, which are designated the experience questionnaire, plan and equipment questionnaire, and the contractor's financial statement.

Leon F. Peck,[7] M. Am. Soc. C. E. (by letter).—The writer fully realizes that a brief should be submitted whereby some lowest bidders should not be awarded the contract. Evidently, this discrimination has been practiced unfairly for many years. It is not new, but those interested are trying to set up a method of making the award with fairness.

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[5] Asst., Dept. of Civ. Eng., Ohio State Univ., Columbus, Ohio.

[6] Acts, 1929, Chapter 202, p. 680.

[7] Supt., Dept. of Streets, Hartford, Conn.

The fact that engineers are discussing means of doing this would indicate that there might be some question as to whether it can be done fairly both to the contractor and to the public, where public funds are involved.

This is a big step, seemingly in the opposite direction from that recommended in 1912 by Maj. Cassius E. Gillette, then President of the American Society of Engineering Contractors which had an active existence over a period of a few years before the World War interruption.

Major Gillete stated in a prepared paper,[8] that he had made investigations of three sets of public contracts and found a loss of $8 600 000 out of a total of $18 000 000 worth of contracts. He said it was partly due to the clause in specifications granting discretion to the awarding officials. He also stated that in all these cases, covering dozens of contracts, the invariable accompaniment of the cheat was the scarcity or absence of real competition, and that under the shelter of that potent word, "discretion," fraudulent and unwise public contracts had been let and interpreted so as to cause the loss to taxpayers of the United States of untold millions, and to rob hundreds of bidders of their just rights.

Major Gillette believed that the American Society of Engineering Contractors should sponsor a bill in every State Legislature in the United States, making it compulsory that no request for bids on public work should be advertised until the appropriation was available, and that the contract must be awarded to the lowest bidder. He had had such wide experience and had given this subject such prolonged study, that it would be unfair not to give his views cognizance at this time.

C. L. HALL,[9] M. AM. SOC. C. E. (by letter).—This interesting, but all too brief, discussion of the merits and demerits of pre-qualification avoids the question of the political factors involved. The construction of public works, like any other act of the State, is necessarily political in origin and in final control. It is imperative, therefore, that the political effect of any phase of public construction be considered. The writer desires to point out that political history may throw some light on the letting of contracts.

All discussion of pre-qualification is based on the theory that, in general, the owner who is to use the work to be built can best protect his own interests by restricting competition to contractors whom he knows are capable of doing the work. In engineering matters, the wise owner follows the advice of an engineer whom he has selected. Usually, the elimination, through error, of a qualified contractor will cost the owner only the comparatively small sum of money representing the difference between the successful bid and the bid which the eliminated contractor would have submitted. On the other hand, even with excellent inspection, an unqualified contractor can sometimes cause an owner very large losses. In almost every case the relatively slight sums he can save the owner by his lower bid at least are balanced by the diminished value of any structure upon which the construction work has been below par, or by the increased cost thereof, if the inspection cost has been unduly high.

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[8] "Competition in Public Contracts", *Journal*, Am. Soc. of Eng. Contractors, Vol. 4, Pt. 1 (1912), p. 87.

[9] Lt.-Col., Corps of Engrs., U. S. A., U. S. Engr. Office, Cincinnati, Ohio.

If the odds are thus weighed, and the owner is assumed to be capable of exercising an intelligent care of his own money, an iron-clad case can be made out for pre-qualification.

In works of any magnitude, however, the owner is usually an abstraction, such as a corporation. In public works, the owner is so much of an abstraction, that he cannot really be identified at all. In order to visualize him those who have dealings with any particular branch of the owner government as to any work, usually refer to the bureau or State commission in charge as the owner. These organs do not resemble very closely the wise private owner.

During the scandals of reconstruction, a half century ago, the contracting officer did not usually yield to crude bribery, but he frequently exercised his official discretion for the benefit of the political organization which, rather than the abstract public, was responsible for his appointment. He thus tended to award contracts for the benefit of contractors, who presumably rewarded the political organization with a share of the profits. It was the scandals that followed this relationship which caused the passage of the strict laws relative to open bidding on public works, now generally in force both in the Union and in the States.

Federal specifications, and most State specifications, even in States that have been most conservative in modifying the general laws, give a contracting officer the right to reject the bid of a contractor who lacks the requisite plant or experience. Rejection after bids are opened, or post-qualification, however, involves many inconveniences to the contracting officer. He may be accused of favoritism, or even of corruption. If the protesting bidder has, or can arouse, political or press influence in his behalf, the officer may be reversed by his political superiors. It is simpler to prevent such difficulties by pre-qualification.

With the present improved standards of honesty among public officials, there is no doubt that pre-qualification does little harm at this time; but it is only human nature for contractors to form a trade association in their own behalf. They have, assuredly, many data to support claims that inexperienced contractors injure the public. Once such an association is formed, it will also be human nature for it to grant new admissions reluctantly. If the association allies itself with employee bodies containing a large number of voters, and if the contractors play politics, the closed contractor list may become a reality.

Professor Tilden's conclusions point out some of the dangers of the closed list, even on the assumption that contracting officers act with complete independence, and in perfect good faith; but complete independence is rarely possessed by non-political officers; nor is good faith always a quality of political appointees. Moreover, the permanent officer, who is most likely to possess both these desirable characters, is by nature a conservative. He prefers to deal with contractors whom he knows. Everything thus conspires to keep a closed list closed.

Humanly speaking, it is certain that a closed contractor list will cause a repetition of the very evils that strict public works laws were designed to suppress. It is at least probable that pre-qualification will produce such a list. Post-qualification in the light of day, as is the present practice of the Federal Government, does not cause these evils. This system is possible if the State has a permanent engineering force, with an *esprit de corps* of its own, and if local public opinion refuses its support when the engineer is over-ruled by the politician on technical matters. These may be counsels of perfection; on the other hand, without such a public organization, it will be impossible to prevent scandals following pre-qualification. The writer, therefore, concludes that statutory pre-qualification would cause disadvantages outweighing its undeniable advantages.

A. R. LOSH,[10] ASSOC. M. AM. SOC. C. E. (by letter).—This is a timely subject and one worthy of careful consideration. The large volume of public works contracts now under way (1932) and the dependence of the construction industry upon such contracts, make it necessary that the best business principles shall prevail. An efficient, qualified, responsible contractor is just as essential as an engineering organization with the same qualifications. Professor Tilden has presented the general features of pre-qualification in an excellent paper, and the writer would like to add a few brief comments, particularly with reference to the advantages of pre-qualification.

There appear to be a number of advantages not brought out in sufficient detail by the author. The administrative authorities on public works are relieved of the responsibility and inconvenience of assisting in collecting unpaid bills from contractors. They are relieved from the criticisms which always follow in the trail of a bankrupt contractor, of merchants, material men, laborers, and others who may sell their goods or services to the contractor. Many people who are not informed as to the provisions of public contracts and surety bonds, will sell contractors or their employees almost anything, feeling that the State or other public authority will protect them. They also feel that an organization that is qualified to receive a contract on public work must have a first-class credit rating.

Surety bonds usually protect the State in securing work within the specifications and time limits, but they do not fully protect the public from losses due to the failure of the contractor to pay his bills. Laborers and small businesses cannot afford to employ attorneys and go into Court to collect their accounts. There are occasions without number where contractors, and subcontractors, or employees, buy goods or obtain services that are not directly related to public contracts, but the uninformed public is not aware of these conditions and is without protection.

Surety companies occasionally fail, and the public is left without protection. The public official cannot specify the surety, but must accept any that is offered as long as it has a satisfactory rating. All surety companies do not handle contractors' claims on the same basis.

Companies that sell materials and equipment will be assured of receiving payments for their goods and will compete for business, resulting in a lower cost to the public. Contractors in good financial standing, who are in position to discount their bills, frequently buy cheaper than the State, while those contractors whose credit is in doubt, must pay a higher price.

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[10] Mgr., Asphalt and Road Oil Div., Anderson-Prichard Corp., Oklahoma City, Okla.

Pre-qualification will have a stabilizing affect on the construction industry which, in turn, will permit the development of better organizations, more efficient work, and, in the end, reasonable prices. Certainly, on public work, prices should not be unreasonably high, or low. Fair prices for materials, living wages, a reasonable profit for the contractor, and the work done as required by the plans and specifications, are the desirable ends to be obtained in the termination of a public works contract.

On the side of disadvantages of pre-qualification, the need is only to guard against restricted competition. To offset this there is always left open to the administrative authority the threat to do the work with its own organization if the bids exceed the engineer's estimate.

E. G. Walker,[11] M. Am. Soc. C. E. (by letter).—The wisdom of adopting some form of pre-qualification is apparent. Stripped of its special signification, the author's statement to this effect merely expresses a fundamental principle of commercial dealing which recognizes that price is only one of the guiding considerations in entering into a business transaction. Unfortunately, the plain common sense of a private deal has a tendency, when applied to the control of public undertakings, to become involved in an elaboration of laws, rules, and red tape. While these are devised primarily for the avoidance of the possible temptation of unethical practice, they also have the tendency to replace the responsible exercises of judgment by the mechanical application of rules. There cannot be any doubt of the practical necessity of a preliminary assessment, by the engineer responsible for the project, of the suitability of the proposed contractor, but there is grave doubt as to the propriety of working strictly by rule in making such assessment. There are many qualifications and attributes of a contractor which may make or mar the execution of a contract, and which are not susceptible of expression in a questionnaire. It is not desirable, therefore, to deal with this matter by formal rules of procedure.

It is the essence of public control that there must be public knowledge of the intention to let contracts for public works and services, and that the terms of these contracts must be fulfilled in the manner that is most economical to the community. The public interest, however, is not served by awarding contracts to the lowest bidder irrespective of other considerations. Hence, there must be some form of limitation in order that the responsible authority may fulfill its function of obtaining the best results for the public it represents. It should be advised, therefore, by a competent engineer who should be free to investigate the claims of competing contractors, and to make recommendations supported by reasoned arguments. In considering his report, the engineer should be free to make such inquiries as appear germane to the issue, and to form his opinions without the limitations of a schedule. In spite of the development of the large contracting company with its complex and sub-divided organization, the successful execution of a public works contract is, to-day, as much dependent upon the individualities of the men in control, as it was in the days of the private individual who made a personal contract for executing works. This must be taken into account adequately in comparing the proposals of competing contractors.

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[11] (Maxted & Knott), London, England.

The prime duty of the engineer is to endeavor so to serve his client as to obtain the best possible results for him within the limitations of the proposition. In order to do this effectively, the engineer has frequently to settle details which involve the striking of a fair balance between the claims of the client and those of the contractor. To do so successfully an evenness of judgment must be developed, which, if properly applied in the initial stages of the making of the contract, will go a long way to facilitate the execution of the work for all concerned therewith. After some years of experience of both public and private works, the writer submits the following three suggestions:

1.—Where the nature of the work and of the interests for whom the work is to be executed, makes public advertisement of the proposed contract essential, it should be open to all contractors to tender; but essential safeguards should be inserted to make clear that the work will not necessarily be awarded to the lowest or to any bidder and, by the requirement of suitable deposits and securities by contractors, precaution should be taken to insure that only proper and serious bids are made.

2.—In cases where public announcement of the proposal is not necessary, it should be within the power of the engineer to invite an agreed number of selected contractors to bid, the engineer having first satisfied the authority responsible for the work as to his reasons for selecting the particular contractors.

3.—In any case, after bids have been received, it should be the duty of the engineer to make such inquiries in any direction as he should think fit, in order to assure himself of the suitability of the contractor before recommending the acceptance of his bid. Naturally, such an investigation would have to be comprehensive or simple, according to the knowledge available from previous transactions. It is essentially a matter in which the procedure in each individual case must be evolved at the time, and cannot be met by the filling in of a pre-arranged form.

It does not seem that the adjudication on the suitability of a contractor to undertake work for which he may compete, can be controlled in detail satisfactorily by legislation, with all the rigidity and limitations which must inevitably accompany it. The subject should be treated in a far more elastic manner. The most suitable treatment is obtained by the employment of competent and honest engineers, and by leaving them full responsibility as outlined herein, subject to the control of the responsible authority. The code of ethics of the Society is a sufficient safeguard to any community served by competent qualified engineers.

C. J. TILDEN,[12] M. AM. SOC. C. E. (by letter).—In closing the discussion of this paper the writer wishes first to review briefly the way in which it came to be written. The Highway Division of the Society chose the subject as the topic for discussion at its meetings in 1930. Two meetings were held, one in April and one in October, at which papers were presented and discussed. Fifteen gentlemen stated their views either in writing or orally. The writer was asked to summarize this material and get it in shape for publication. A paper was published in *Civil Engineering*, for March, 1931, in which the real authors of the views expressed were named.[13] Once again the task was imposed of further condensing the article for publication in *Proceedings*. The present paper was the result. Even a foster parent has an affection for his child, and it is a source of great pleasure to the writer that so many have taken the trouble to discuss it.

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[12] Strathcona Prof. of Eng. Mechanics, Yale Univ., New Haven, Conn.

Mr. Bush, who brings to the discussion the combined wisdom of an engineer and an official of a surety company, makes the interesting suggestion "to state in the call for bids that no bid will be received and opened from a bidder who has not qualified himself for the contract." This would mean a special pre-qualified list prepared in advance for each contract. He urges the desirability of having any scheme of qualification as simple and direct as possible so that good bidders will not be frightened off. He speaks further of the advantage of including in the list of basic elements determining the principles of pre-qualification some measure of the skill and ability of the contractor and the skill and experience of his organization, together with the information about other contracts which he may have in hand. These might be assumed to be included under the general heading of "previous experience" and "character and general reputation," but they are important enough to be mentioned specifically by themselves. Mr. Carpenter also stresses this point.

Mr. Peck quotes the stand taken by Maj. Cassius E. Gillette about twenty years ago, at a time when he was President of the American Society of Engineering Contractors, an organization which appears to be no longer in existence. The views which Major Gillette held in 1912, as a result of careful consideration of the whole question, are nevertheless interesting reading and worthy of note.

Colonel Hall has presented a singularly clear and able brief for the opposition; that is, emphatically against the principle of pre-qualification. In all, about twenty-two individuals have given their views, including those who presented the first group of papers and discussions. Of these, fourteen express themselves in favor of the general principle of pre-qualification; six are noncommittal or deal in their discussion with other phases of the question; while only two are opposed. One of these is Mr. Peck, who quotes Major Gillette as being (in 1912) opposed to the principle; the other is Colonel Hall. Colonel Hall, then, is the emphatic representative of the case against pre-qualification. His discussion is so clear and excellent in every way that it should be read by all who are in any way concerned with the application of the principle. His conclusion, as stated in the last sentence of his discussion, is that "statutory pre-qualification would cause disadvantages outweighing its undeniable advantages."

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[13] "A Summary of the Present State of Prequalification," *Civil Engineering*, March, 1931, p. 504.

On the other hand Mr. Losh thinks, that the advantages of pre-qualification are not brought out in sufficient detail in the original paper, and he proceeds to cite some of these advantages. He calls attention especially to the necessity of guarding against restricted competition.

Mr. Walker, writing from London, England, is mildly in favor of pre-qualification, but thinks it should be applied with modifications which he sets forth under three heads.

In general summary then, there seems to be a decided majority in favor of the basic principle of some form of pre-qualification of prospective bidders on public work. Just what form it should take appears to be subject to a rather wide range of opinion. Whether a generally satisfactory statement looking toward statutory principles could be written at this time is a matter of doubt.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1862

TESTS FOR HYDRAULIC-FILL DAMS[1]

By Harry H. Hatch,[2] M. Am. Soc. C. E.

With Discussion by Messrs. Charles H. Paul, Joel D. Justin, Jephtha A. Wade, Stanley M. Dore, D. P. Krynine, M. M. O'Shaughnessy, Joel B. Cox, William P. Creager, and Harry H. Hatch

Synopsis

This paper describes some of the tests made for the City of Springfield, Mass., on the Cobble Mountain Hydraulic-Fill Dam. The derivation is given of almost all the formulas used in the paper. In addition to new formulas and data pertaining to tests, the paper contains a consolidation study of core materials and a seepage formula for hydraulic-fill dams. It also gives the results of the gradation tests made at Cobble Mountain Laboratory of the core materials of several other hydraulic-fill dams.

Its main object, however, is to arouse interest among engineers connected with hydraulic-fill construction, with a view of standardizing methods of testing materials in hydraulic-fill dams. Opinions differ; engineers on each job devise a different method of testing; and it is time that a definite step is taken and a standard method developed in making tests for this particular kind of construction.

The original manuscript, which is several times the length of this paper, is on file for reference in Engineering Societies Library, 33 West 39th Street, New York, N. Y. It contains forty-one illustrations, including curves of many of the equations.

Purpose of the Laboratory

Hydraulic-fill dams are almost new compared with other types, and they are becoming more popular because of their many advantages. Unfortunately, methods of testing materials for such dams have not been standardized. No doubt the materials for different dams at different locations are not the same, and many dams have local problems; but in the main a standard method could be devised, such as is being used for cement and other materials of construction, and, whenever necessary, this method could be modified for local conditions. After all, the requirements of the core and beaches of a hydraulic-fill dam are the same for any dam and at almost any location.

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[1] Presented at the Joint Meeting of the Irrigation and Power Divisions, Yellowstone National Park, Wyoming, July 7, 1932.

[2] Engr. in Chg., Cobble Mountain Reservoir, Springfield Water-Works, Blandford, Mass.

Heretofore, the methods of testing the borrow and core materials of hydraulic-fill dams have not been along the same lines. As a rule, the ordinary sieve analyses have been made with the statement that a certain percentage of the material is finer than a 100-mesh or 200-mesh sieve. Often the sieves are not even rated; for sieves finer than the 200 mesh the methods differ greatly, and as could be expected, the results are not always entirely reliable. The purpose of Cobble Mountain Laboratory was to make all the tests necessary for this kind of construction and also to establish a satisfactory, and yet practical, method of grading the material finer than the 200-mesh sieve to at least 0.01 mm.

The fact that the laboratory was established on the job proved of great advantage in keeping a close control of the borrow and sluiced materials of the dam.

SAMPLING

In making the preliminary studies on borrow-pits for the determination of the percentage of a critical size and also for making the gradation curve of the borrow material, one must have a representative sample of the pit. Since the gradation curve must show the material that will go over to the dam, it should not include the large boulders in the pit that will be left behind.

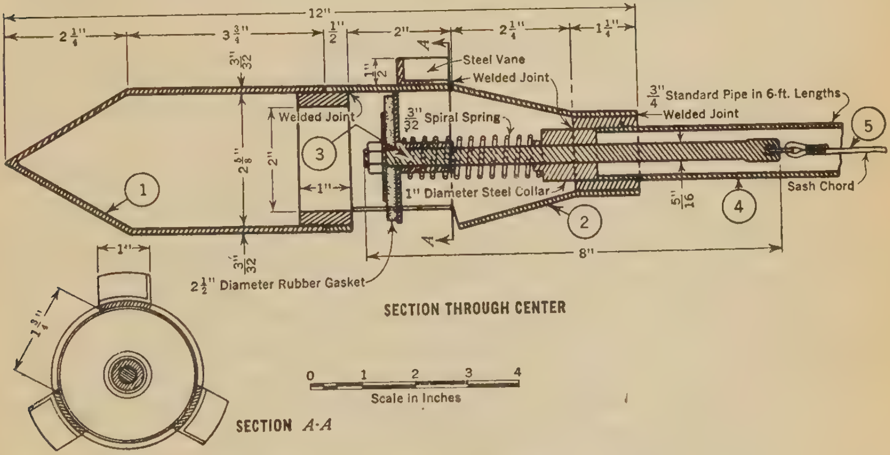
12"
2 1/4"
3 3/4"
1/2"
2"
2 1/4"
1 1/4"
3/32"
1/2"
A
Steel Vane
Welded Joint
3/4" Standard Pipe in 6-ft. Lengths
Welded Joint
5
Welded Joint
3/32" Spiral Spring
3
2 5/8"
2"
1"
1
1" Diameter Steel Collar
5/16"
4
Sash Chord
2
A
8"
1"
3/32"
2 1/2" Diameter Rubber Gasket
SECTION THROUGH CENTER
1 3/4"
0
1
2
3
4
Scale in Inches
SECTION A-A

FIG. 1.—CROSS-SECTION OF PROJECTILE SAMPLER.

Particular care and special equipment are required to secure representative samples from the core of a dam during its construction. At Cobble Mountain the so-called projectile sampler, designed by the Staff of Murray and Flood, Engineers, shown in Fig. 1, was used. Essentially, it consists of: (1) A detachable container for the sample (about 400 cu. cm.); (2) a casing containing the plunger-valve spring, with three openings for admittance of the sample material; (3) a valve and spring operated by pull on the sash cord; (4) a standard, ¾-in. pipe; and (5), the sash cord extending to the surface of the core pool. The sample, taken out with the projectile sampler from a more or less consolidated core, will be of no value for the determination of moisture content or percentage of voids. It is almost impossible to bring out the sample in its original condition with this device.

The following method was found to be satisfactory for taking out samples in their original condition from a consolidated core. One 2-in., or one 1½-in. pipe (or a series of them coupled to any length), having the bottom covered with a canvas, was lowered to a desired depth by means of additional weights. After making sure that there was no water at the bottom of the pipe, a ¾-in. pipe was lowered inside the larger one. The smaller pipe had a piece about 1 ft. long at the bottom, which was beveled to a cutting-edge. The smaller pipe was forced down (the cutting-edge piercing the canvas) into the core about 1 ft., or more. Then it was raised, and its lower end section was uncoupled. The sample was forced out by a wooden piston, and was immediately brought to the laboratory where the moisture content and percentage of voids were determined as soon as possible.

Specific Gravity

The specific gravity of the materials is of great importance. After the value of the specific gravity is determined definitely, it should be checked occasionally to see whether there is any variation. The tests at Cobble Mountain Dam were made essentially as described[3] in the method for testing cement of the American Society for Testing Materials.

The Percentage of Voids

The percentage of voids and moisture content are related, and they give a good idea of the consolidation of the core. The late Allen Hazen, M. Am. Soc. C. E., stated[4] that,

> "The percentage of voids furnishes, on the whole, the best index of consolidation and stability. In itself, it is not an adequate basis of comparison, because different kinds of core material may have different degrees of stiffness with the same percentage of voids; but, notwithstanding this difference, the percentage of voids is the best index of consolidation so far available."

The determination of the percentage of voids, if it is done accurately, is essential. It is readily seen that the unit weight (and, therefore, the acting and resisting forces) and the coefficient of permeability of the different parts of a dam are directly dependent on, and the coefficient of friction is affected by, the percentage of voids of its material.

If at any depth and location in the core a sample in its original position with its original moisture content can be brought to the surface, the moisture content of the core in the dam at that particular elevation can easily be computed. In saturated core material all voids are filled with water and, when the specific gravity of the material is known, the percentage of voids can be computed by the formula,

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[3] Specifications C9–26, Am. Soc. for Testing Materials.

[4] "Hydraulic-Fill Dams", by Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 1718.

$$V = \frac{w}{w + \dfrac{(100 - w)}{s}} = \frac{100\ ws}{ws + (100 - w)} \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, V equals percentage of voids; w, percentage of water by weight; and, s, specific gravity of core material. The computation is simple and can be made readily, or the equation can be interpreted into a set of curves from which values can be picked.

Two methods were used in the determination of the percentage of voids in the beach materials—the one for the beach material near the core limits, where the particle sizes are small, and the other for the beach material near the toes, where the particle sizes are large.

For beach materials with particle diameters up to about ¼-in., brass cylinders, 4 in. in diameter and 6 in. long, were used. One end of the cylinder was beveled to a cutting-edge to facilitate forcing it into the beach. Each cylinder was pressed into the beach and then removed with a shovel having considerable material above and below it. The top and bottom of the sample were neatly cut with wire, giving a sample equal in volume to that of the cylinder. The material was then dried and weighed, and the percentage of voids computed.

For larger particles near the toes of the beach, a steel pipe, 20 in. in diameter and about 3 ft. long, open at both ends, was used. At places where the samples were taken the surface material was scraped down to a level and undisturbed section in the beach. The pipe was forced down by sledge hammer only 4 or 5 in. at a time; then the inside material was cleaned out and set apart. Later, it was forced down a similar distance, and the operation continued until about 4 cu. ft. of sample material was obtained. After it came out of the pipe, the test material was weighed in the field and a sample was brought to the laboratory for moisture-content determination. With this information the dry weight of the sample in the field was computed and the percentage of voids determined.

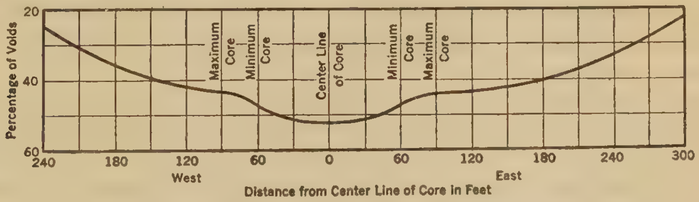

FIG. 2.—PERCENTAGE OF VOIDS ON A SECTION OF COBBLE MOUNTAIN DAM AT DIFFERENT DISTANCES FROM THE CENTER LINE OF THE CORE.

It was noted that the percentage of voids was at its maximum in the core pool, decreasing toward the toes (Fig. 2). The reason for this may be due to the fact that the beach material is deposited by running water and that the core material held in suspension is deposited by settling in comparatively still water. Furthermore, the material close to the core limits is more or less uniform in size, whereas farther away from the core section the beach material becomes larger, has a better gradation, and, therefore, a smaller percentage of voids.

The percentages of voids shown on Fig. 2 are not final and are good only as a comparison in showing the variation in consolidation of the various points in the dam at or about the same elevation. Undoubtedly, due to increased weight above, the consolidation shown in Fig. 2 will be somewhat increased as the dam is completed and as the final settlement takes place.

PENETRATION TESTS

A method previously used has been to make the penetration tests in the field by two strong men forcing down one 1½-in. pipe into the core as far as it will go. It has been stated that when the pipe is forced down, the core material at the elevation of the bottom of the pipe has 50% voids. The definition of "two strong men" is rather uncertain, and it is doubtful whether 50% voids could be obtained for any two materials at the elevation of the bottom of the pipe, even if the force exerted by "two strong men" was constant and known. Experience shows that the penetration of the pipe is affected not only by the percentage of the voids in the core, but also by the grain sizes and other physical characteristics of the core material.

To obviate any uncertainty in regard to the force necessary to reach the penetration limit on the 1½-in. pipe, definite loads up to 600 lb. were applied to sink it. A complete operation was conducted for each loading. In some cases, the bottom of the pipe was tied with greased canvas to prevent mud and water from getting into it. However, it was found that the depth of penetration was not materially affected by the canvas at the bottom of the pipe. When the penetration limit was reached for any loading, the weights were removed, and samples were taken from the core by the ¾-in. pipe, as described under "Sampling," at the elevation of the bottom of the 1½-in. pipe, and the percentage of voids of the material was computed.

It was discovered that by applying the total load at the beginning of the test, the pipe would penetrate farther into the core than by applying the same total load in fractions at different intervals during the process of penetration, when it took time to make the necessary additional connections and to increase the loading. In every instance, the latter case gave about 2 or 3 ft. less penetration.

It is evident that there is a distinct relation for the same material between the total load applied to the 1½-in. pipe and the percentage of voids of the core material at the elevation of the bottom of the pipe. Once this relation is established for any one material, it will not always be necessary to determine the percentage of voids for every test.

Notwithstanding the personal equation both in the field and in the laboratory, the difference in physical characteristics, and the difference in the grain sizes of the core material, the numerous tests gave the following approximate relation between the percentage of voids of the material and the total load causing the penetration:

$$100 - V = 33\,L^{0.067} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which, V is the percentage of voids, and L, the load, in pounds, including the weight of the $1\frac{1}{2}$-in. pipe. As in the case of Equation (1) this formula can easily be expressed in curve form for easy reference. In winter, it was more convenient to carry on this investigation by building a timber tower over the ice on the core pool, in order to facilitate the operations necessary for conducting the test.

RATING SIEVES

The mesh openings of a sieve given by the manufacturer, or their exact measurement, will not give the actual separation sizes of the material. Certain rules have been established to apply to the nominal mesh openings of a sieve in order to obtain its actual separation; but it is desirable to determine the actual separation of the sieves used on the job for the particular material on hand. The shape and other characteristics of a material will affect the value of the separation. A sieve will not necessarily have the same separation size for different materials. In fact, rating of the same set of sieves should be repeated from time to time as the meshes will wear and some of the finer meshes will get clogged.

The 200-mesh sieve is the most unreliable in this respect; it requires constant watching. The rating of sieves on the Cobble Mountain Dam was conducted along the principles outlined[5] by the American Water Works Association. The variation in separation sizes is shown in Table 1. The No. 3 rating was checked by microscopic examination, and its variation was found to be due to differences in the shapes of particles.

TABLE 1.—THE VARIATION IN SEPARATION SIZES OF THE TYLER SIEVES USED AT COBBLE MOUNTAIN

Mesh No.	Mesh opening, in millimeters (Tyler values)	SEPARATION, IN MILLIMETERS, BY RATINGS					Separation values used
		No. 1	No. 2	No. 3	No. 4	No. 5	
4.............	4.699	5.02	5.03	....	....	....	5.02
8.............	2.362	2.60	2.60	....	....	....	2.60
14.............	1.168	1.29	1.29	....	1.27	....	1.27
28.............	0.589	0.65	0.63	0.647	0.62	....	0.62
48.............	0.295	0.33	0.32	0.327	0.31	....	0.31
100.............	0.147	0.16	0.16	0.166	0.15	....	0.15
200.............	0.074	0.086	0.082	0.086	0.095	0.09	0.09

ELUTRIATION

A sieve analysis with a minimum mesh of 200 is limited to about 0.09 mm. in diameter, whereas it was necessary to carry an analysis to at least 0.01 mm., or about one-tenth the particle sizes of the 200-mesh limit.

Elutriation by the decantation method, as described[6] by F. E. Turneaure and H. L. Russell, Members, Am. Soc. C. E., has been used to analyze sand that passes a 200-mesh sieve. The tests at Cobble Mountain Dam were made to control the sizes of the core material during construction. The method cannot be practical because of the lack of automatic control in operations and because of the many personal equations entering into the test, not to mention the slow process. Hence, it became necessary to develop a practical method that would take a reasonably short time to make tests of a few samples a day with reliable results, and, therefore, a practical apparatus for elutriation was designed, and grading by elutriation was adopted.

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[5] Manual of Water Works Practice, 1925, p. 641.

[6] "Public Water Supplies", by F. E. Turneaure and H. L. Russell, 1910, p. 472.

Various types of elutriators[7] have been used in laboratories for some time, some of them dating back to about 1900. For the first time, at Cobble Mountain Dam, an elutriation method was used in a practical way for the control of the core material of a dam under construction.

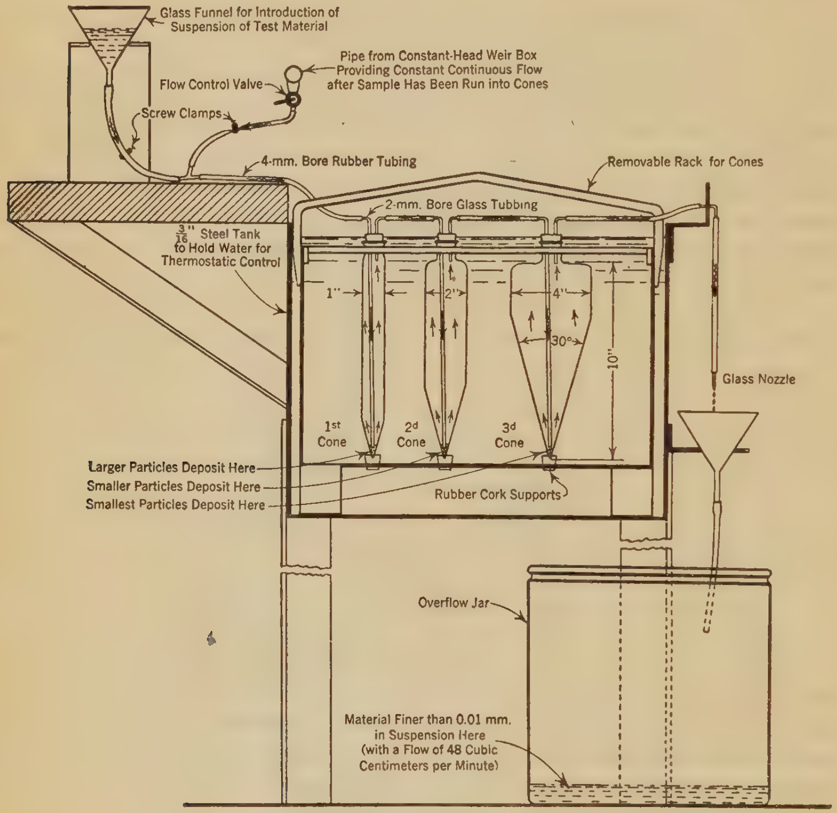
Glass Funnel for Introduction of Suspension of Test Material
Pipe from Constant-Head Weir Box Providing Constant Continuous Flow after Sample Has Been Run into Cones
Flow Control Valve
Screw Clamps
4-mm. Bore Rubber Tubing
Removable Rack for Cones
2-mm. Bore Glass Tubing
3/16" Steel Tank to Hold Water for Thermostatic Control
1"
2"
4"
30°
10"
Glass Nozzle
1st Cone
2d Cone
3d Cone
Larger Particles Deposit Here
Smaller Particles Deposit Here
Smallest Particles Deposit Here
Rubber Cork Supports
Overflow Jar
Material Finer than 0.01 mm. in Suspension Here (with a Flow of 48 Cubic Centimeters per Minute)

FIG. 3.—ONE UNIT OF ELUTRIATOR TO SHOW OPERATION.

The apparatus (see Fig. 3) consists of three glass cones of different sizes, each provided with an inlet tube running to the bottom and an outlet tube running to the next cone. The first cone inlet tube has a connection to a glass funnel into which the test material is first placed. From this funnel, the material is carried to the first, then the second, and then the third, cone which has a connection to a glass nozzle that discharges the overflow into a large glass jar.

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[7] "Clays", by Heinrich Ries, 1927, pp. 198–202.

With the arrangement of the inlet tubes, a rising column of water is created in every cone, each with a different velocity, due to the different cross-sections of the cones. When the sample is washed through the glass funnel into the first cone only the larger particles settle because at this point the water has its greatest velocity. The smaller particles are carried over into the next cone. Similarly, the same action takes place in the other cones, the particles settled in them having a definite relation to their areas. The test is continued until the overflow discharge becomes almost clear. At the end of the run the particles at the bottom of each cone are recovered by a process of filtration through alundum crucibles under a vacuum, and the overflow is flocculated with chemicals and then filtered.

The Cobble Mountain elutriator has four units, so that four tests can be run at the same time. All the units are submerged in water in a steel tank. The flow of the water through the cones is kept constant by a weir box, in which the head is constant, attached to the tank. The thermostatic control and the circulating pump keep the water at a constant and uniform temperature throughout the tank and in the cones. With a correspondingly longer tank, almost any number of elutriator units can be installed and as many tests run at the same time.

THE ELUTRIATOR FORMULA

The elutriator formula is derived from Stokes' law[8]:

$$v = \frac{2}{9}\,\frac{(P_p - P_l)}{n}\,g\,r^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, v is the velocity of fall, in centimeters per second, P_p, density of particle; P_l, density of liquid; g, gravity constant, in centimeter-gramme-seconds $= 981$; r, radius of particle, in centimeters; and n, coefficient of viscosity, in poises.

For working purposes, the flow through the elutriator tubes should be expressed in cubic centimeters per minute and the diameter of the particle in millimeters. Then, $60\,v = 60 \times \frac{2}{9}\,\frac{(P_p - P_l)}{n}\,981\left(\frac{d}{2 \times 10}\right)^2$; but, $60\,v = \frac{\text{cubic centimeters per minute}}{\text{square centimeters}}$; therefore,

$$\frac{\text{Cubic centimeters per minute}}{\text{Square centimeters}} = \frac{60 \times 2 \times 981}{9 \times 4 \times 100}\left(\frac{P_p - P_l}{n}\right)d^2$$

$$= 32.7\left(\frac{P_p - P_l}{n}\right)d^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Without appreciable error values of specific gravity instead of those of density can be substituted for P_p and P_l. Let $P_p = 2.72$ for Cobble Mountain material, and $P_l = 1.00$. Then, $P_p - P_l = 1.72$. Substituting this value in

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[8] Mathematical and Physical Papers, by G. G. Stokes, Vol. III, p. 60, Cambridge Univ. Press, 1905.

Equation (4) and solving for d,

$$d = 0.13334 \sqrt{\frac{\text{cubic centimeters per minute}}{\text{square centimeters}}} \sqrt{n} \quad \ldots\ldots\ldots\ldots (5)$$

which is the elutriator formula. By inserting the actual values of the flow, area of the tube, and the coefficient of viscosity, in Equation (5), the diameter of separation can be computed in millimeters.

Instead of the viscosity coefficient, it is proposed to use the temperature factor, $\frac{60}{t+10}$, in degrees Fahrenheit. The relation between the coefficient of viscosity and $\frac{60}{t+10}$ is not strictly constant (see Fig. 5); the relation factor varies from 0.01274 to 0.01311 for a range of temperature from 40° to 90° Fahr. The average value of 0.01296 was adopted to apply to the range of working temperature for this factor; thus,

$$n = 0.01296 \left(\frac{60}{t+10}\right) \quad \ldots\ldots\ldots\ldots (6)$$

Substituting for n in Equation (5) the value from Equation (6) and making the necessary reduction,

$$d = 0.0152 \sqrt{\frac{\text{cubic centimeters per minute}}{\text{square centimeters}}} \sqrt{\frac{60}{t+10}} \quad \ldots\ldots\ldots\ldots (7)$$

Equation (7) is the one used at Cobble Mountain Reservoir. Eleven microscopic measurements were made at the hydraulic-fill laboratory, and their values for the constant, 0.0152, in Equation (7) were: 0.0155, 0.0154, 0.0172, 0.0163, 0.0157, 0.0155, 0.0157, 0.0151, 0.0165, 0.0151, and 0.0161. The average of these eleven values is 0.0158 compared with 0.0152 as computed. The difference, which is probably due to shape of particles, is negligible, and the two results agree satisfactorily.

Stokes' law gives the constant velocity of falling bodies with a certain diameter through a liquid, whereas, in this case, the same law applies to bodies of the same diameter held in suspension by, or resisting the upward flow of, the same liquid, having, of course, the relative values given in Equation (7).

In order to obtain reliable results for a test, it is necessary that the flow (in cubic centimeters per minute), and the temperature (t° Fahr.) factors of Equation (7) should remain constant during the run of the test. The weir box furnishes a constant head, and a desired uniform flow can be had throughout the run from a small quantity to about 80 cu. cm. per min. The heat control can be set to a desired temperature, and it will work automatically to keep the water at that constant temperature. The circulating pump, of course, will insure a uniform temperature for the water throughout.

The net cross-sectional areas of the cones are: $A_1 = 4.65$; $A_2 = 20.81$; and $A_3 = 85.00$ sq. cm. With a flow of 48 cu. cm. per min. and $t = 77°$ Fahr., Equation (7) will give, to nearest 0.01 mm.:

$d_1 = 0.04$ mm. for first stage, the smallest cone.
$d_2 = 0.02$ mm. for second stage, the middle cone.
$d_3 = 0.01$ mm. for third stage, the largest cone.

It is desirable to keep the separation sizes constant; and in case of any change in the temperature, the flow can be adjusted to offset this temperature effect, thus giving a constant, d, in Equation (7). It is seen from the formula that d varies as the square root of the flow. Then, by adjusting the flow to desired values, separation can be determined for corresponding smaller sizes.

As a practical procedure, however, this is not advisable. Duration of the test varies inversely as the flow. A flow of 48 cu. cm. per min. will give a lowest separation of 0.01 mm., and the actual run of the test through the elutriator will take nearly four times as long for a separation to a fineness of 0.005 mm. Of course, according to the formula, the law will also hold for higher flows—no doubt within limits—with larger separations and less time.

The results obtained by the elutriator are reliable especially when they are checked now and then by microscopic examination. This method is a great improvement in the accurate and rapid determination of grain sizes finer than the 200-mesh sieve compared with other methods used heretofore.

Microscopic Control

Microscopic measurements may involve assumptions which perhaps are not exact and which may influence the results, and yet it is the most reliable method available for observation and checking purposes. The microscope is very useful also to observe and study the shape and other characteristics of particles.

Periodically, microscopic measurements of the separation of the 200-mesh sieve and also of the three stages of the elutriator were made at Cobble Mountain in accordance with the method outlined[9] by Professor Lincoln T. Work.

The Hydrometer

The most rapid method, with reasonable accuracy, in determining the grain sizes finer than the 200-mesh sieve, known to the writer, is by means of the hydrometer developed by Dr. J. G. Bouyoucos.[10] Within almost an hour, one-third of the minimum time necessary for the elutriator, an analysis can be made of a soil sample by this method. The apparatus required is very simple and its cost is almost negligible compared with that of the elutriator. This method has many advantages, and it is being used in many places as the only means of determining the grain sizes of soil particles.

The hydrometer is so constructed that the graduation on its stem at the water surface will indicate the number of grammes of soil in suspension per liter of water in the graduate containing the hydrometer. As a precaution, however, the readings should be checked with various predetermined quantities of grammes of soil per liter of water, or with solutions of known concentration. The theory upon which this method is based follows also from Stokes' law as given by Equation (3). Substituting the values of $P_p = 2.72$, $P_l = 1.00$, $g = 981$, and $r = \frac{d}{2}$,

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[9] "Methods of Particle Size Determination", by Lincoln T. Work, *Proceedings*, Am. Soc. for Testing Materials, Vol. 28, Pt. II, 1928, p. 780.

[10] "Directions for Determining the Colloidal Material of Soils by the Hydrometer Method", by J. G. Bouyoucos, *Science*, July 1, 1927, Vol. LXVI, No. 1696, pp. 16–17; and "Making Mechanical Analyses of Soils in Fifteen Minutes", by J. G. Bouyoucos, *Soil Science*, Vol. XXV, No. 6, June, 1928, pp. 473–480.

$$d = 0.1033\,(vn)^{\frac{1}{2}} \qquad \text{(8)}$$

Equation (8) can also be used for *d,* in millimeters, with *n,* in centipoises. The value of *n* for any temperature can be obtained from tables of coefficients of viscosity; therefore, it only remains to find the value of *v* for the determination of *d;* thus,

$$v = \frac{\text{Distance, in centimeters}}{\text{Time, in seconds}} \qquad \text{(9)}$$

At different time intervals observations are made of the concentration (in grammes of soil per liter of water) of the suspension at the hydrometer center of buoyancy. From the beginning of the test to the time of any observation, during so many seconds, particles having diameters larger than a certain critical size are traveling by, or particles finer than that critical size are still in suspension above, the center of buoyancy. The traveled distance is taken from the water surface to the buoyancy center, but this is not constant for different concentrations of a suspension; neither is it necessarily the same for any two hydrometers. For each hydrometer a curve similar to Fig. 4 should be prepared. With distances obtained from Fig. 4, and the time, in seconds, as observed, the velocity is computed and Equation (8) is solved for *d.*

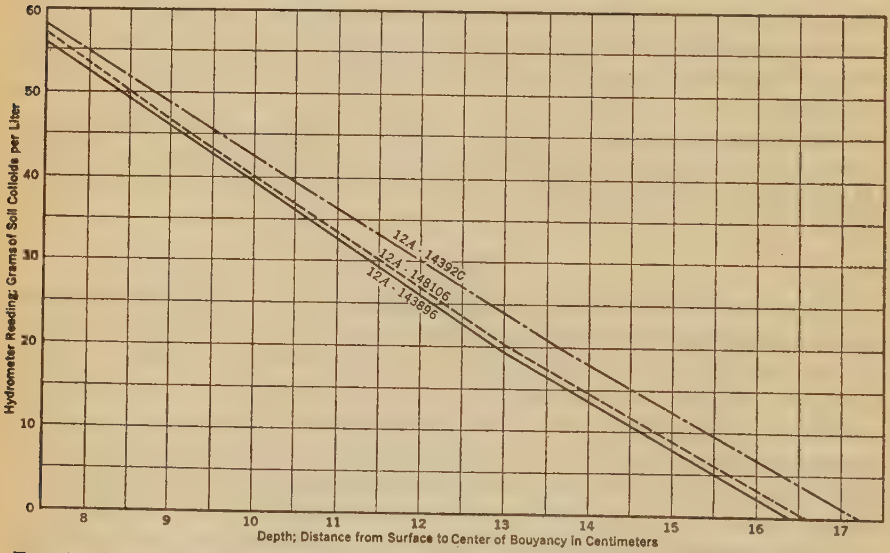

FIG. 4.—CURVES SHOWING DISTANCE FROM CENTER OF BUOYANCY TO WATER SURFACE, PLOTTED AGAINST GRAMMES OF SOIL PER LITER.

The hydrometer has been designed for a water temperature of 67° Fahr. = 19.44° cent. If water is used with another temperature it will be necessary to make a correction due to its change in density according to the formula,

$$R_o = \frac{s}{s-1} 1\,000\,(P_o - P_t) + R_t \ldots\ldots\ldots\ldots\ldots (10)$$

in which, R_o is hydrometer reading at 19.44° cent.; s, specific gravity of soil; P_o, density of water for 19.44° cent.; P_t, density of water at t° cent.; and R_t, hydrometer reading at t° cent.

The term, $\frac{s}{s-1} 1\,000\ (P_o - P_t)$, is the correction to be applied to the reading of the hydrometer at any temperature. The water has a maximum density at about 4° cent., and it decreases gradually as the temperature rises. The density of water below 4° cent.—in fact, below that of the working temperature—need not be considered. Whenever the temperature is below 19.44° cent., the correction will be negative, or with a minus sign, and *vice versa.* The correction term can be plotted for any value of specific gravity, from the relation:

$$C_t = 1\,000 \frac{s}{s-1} (P_o - P_t) \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

As a precaution, however, it is well to take hydrometer readings at different temperatures and check the theoretical correction curve. The corrected hydrometer reading gives the weight of particles finer than those of d for which the observation was made. This weight, divided by the total weight of the original sample, gives the percentage finer than d. With observations made at different time intervals, corresponding computations are completed and the gradation curve is plotted.

Some soils in their natural state contain elements which produce flocculation when the material is placed in pure water. In most cases this may be overcome by the use of a deflocculating agent (such as, ammonia or sodium oxalate). For local materials which showed a very slight, or no, tendency to flocculate, a fixed quantity (5 cu. cm. per liter) was used satisfactorily.

Gradation Procedure by Sieving and Hydrometer Method

The apparatus required for grading material by the sieving and hydrometer method follows:

One gyratory riddle with nest of Tyler sieves (Mesh Nos. 4, 8, 14, 28, 48, 100, and 200);
One 250-mesh sieve;
One motor, cup, and stirring device;
One glass cylinder graduated to about 1 100 cu. cm.;
One hydrometer graduated in grammes of soil colloids per liter of water at 67° Fahr.;
One centigrade thermometer (—10° to 110° cent.);
One stop-watch; and miscellaneous laboratory equipment, including wash bottle, balance, drying oven, etc.

The preliminary treatment entails five steps, as follows:

1.— Determine the moisture content of the moist soil sample to be tested, and weigh out the amount necessary to give about 35 grammes of — 200-mesh material.
2.— Place the sample in the stirring cup, washing out all particles from the weighing container with water; and fill the cup about three-fourths full with water and add 5 cu. cm. of ammonia or sodium oxalate ($Na_2C_2O_4$). Experiments with local materials showed that the 5 cu. cm. of dispersing agent gave satisfactory results in all cases.
3.— Place the cup in the stirrer, and stir for about 10 min.
4.— Wash the material through a 250-mesh sieve, drying the coarse fraction at 105° cent.
5.— Make a sieve analysis of the coarse portion (Sieve Nos. 4, 8, 14, 28, 100, and 200).

The sample is prepared for the hydrometer run in two steps, namely,

6.— Add the — 200-mesh material from the dry sieving to the — 250-mesh material in suspension from the preliminary washing; and
7.— Transfer the suspension to a liter graduate and fill with water to a volume equivalent to 1 000 cu. cm. plus the volume of soil used.

Then, the hydrometer run involves four final steps:

8.— Agitate the suspension by reversing the cylinder about ten times, covering the opening with the palm of the hand.
9.— Set the cylinder on a table and start the stop-watch immediately.
10.— Place the hydrometer in the cylinder.
11.— Read the hydrometer at 1, 2, 5, 10, 30, and 60 min., taking care to keep the hydrometer stem wet so that observations are taken on similar menisci.

Computation.—A computation based on an analysis of Sample *P*-11, Test *H*416, will be used to illustrate this procedure:

Moisture Content, by Step 1:

		Grammes.
Wet material + crucible	=	23.790
Crucible empty	=	12.874
Wet material	=	10.916
Dry material + crucible	=	20.954
Crucible empty	=	12.874
Dry material	=	8.080

$$\text{Percentage of dry material} = \frac{8.080}{10.916} \times 100 = 73.9$$

A total of 80 grammes of wet material was used; therefore, its equivalent dry weight was 80 × 0.739 = 59.12 grammes.

Sieving, by Step 5:

The weight of the + 250-mesh material to be sieved is 40.63 grammes. The analysis of the sample given in Table 2, shows that 40.57 grammes were recovered in the sieving. In other words, 40.63 − 40.57 = 0.06 grammes, lost in sieving.

TABLE 2.—ANALYSIS OF MATERIAL RETAINED ON A 250-MESH SIEVE

Mesh No.	Separation, in millimeters	Grammes retained	Cumulative grammes finer	Cumulative percentage finer
4	5.02	0	.....	100.0
8	2.60	0	.....	100.0
14	1.27	0	59.06	100.0
28	0.62	0.05	59.01	99.9
48	0.31	0.20	58.81	99.6
100	0.15	3.20	55.61	94.1
200	0.09	10.03	45.58	77.1
—200 (dry)	....	27.09	.....	.....
Total sample recovered from sieving	....	40.57	.....	.....

Hydrometer Run, Steps 8 to 11, Inclusive:

The arrangement of observations and computations for Steps 8 to 11, inclusive, is shown in Table 3.

TABLE 3.—OBSERVATIONS AND COMPUTATIONS

Elapsed time, in minutes (1)	Temperature, in degrees centigrade (2)	HYDROMETER READING Observed (3)	HYDROMETER READING Corrected (4)	Distance, in centimeters (5)	Velocity, in centimeters per second (6)	Viscosity, n (7)	Diameter, in millimeters (8)	Percentage finer (9)
1	19.2	27.3	27.2	11.95	0.199	1.025	0.0467	46.0
2	19.2	19.8	19.7	13.10	0.109	1.025	0.0345	33.3
5	19.2	12.0	11.9	14.45	0.0482	1.025	0.0230	20.2
10	19.2	9.0	8.9	14.95	0.0249	1.025	0.0165	15.1
30	19.0	6.0	5.9	15.50	0.0089	1.030	0.0097	10.0
60	19.0	4.3	4.2	15.80	0.0044	1.030	0.0069	7.1

These values (Table 3) are computed, as follows:

Column (1)	Observed
Column (2)	Observed.
Column (3)	Observed.
Column (4)	From Equation (10).
Column (5)	From Column (3), Table 3, and Fig. 4.
Column (6)	From Columns (1) and (5), Table 3, and Equation (9)
Column (7)	From Fig. 5.
Column (8)	From Equation (8).
Column (9)	Column (4) Table 3, divided by total sample (59.06 grammes).

Plotting.—Plot the hydrometer grading, together with the sieve analysis showing the particle size, in millimeters, against the cumulative percentage passing a given size of sieve. (See Fig. 6.) From the grading curve pick off the 10% (effective) size, and the 60% size and compute the uniformity coefficient, by dividing the latter by the former.

COMPARISON OF ELUTRIATOR AND HYDROMETER METHOD RESULTS

Many comparisons have been made of the results obtained by the elutriator and the hydrometer methods at Cobble Mountain. The hydrometer gradation curves, in the lower limits, are invariably above those of the elutriator, thus showing a higher percentage of finer particles. (See Fig. 6.) Logically, it

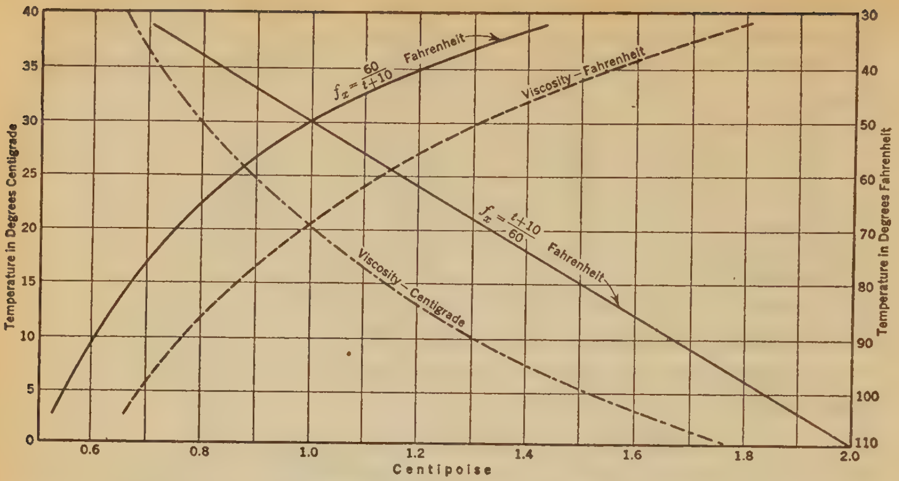

FIG. 5.—RELATION BETWEEN VISCOSITY AND TEMPERATURE FACTOR.

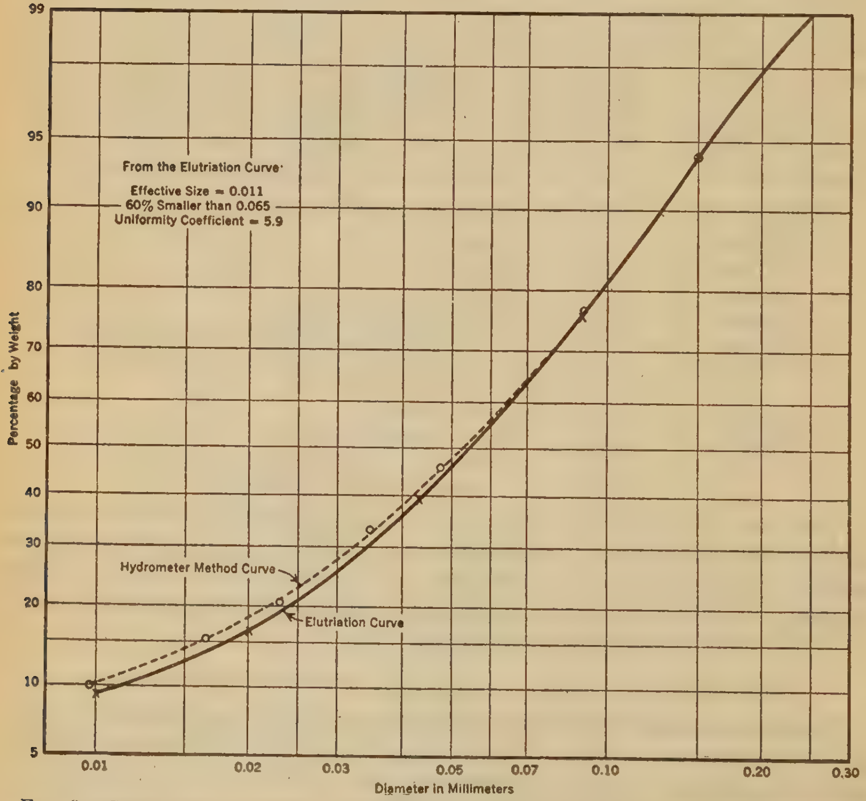

FIG. 6.—COMPARISON OF GRADATION BY THE ELUTRIATION AND HYDROMETER METHOD.

should be the reverse, as one would think that some of the fines would undoubtedly go down with the larger particles, thus resulting in a coarser curve. Sometimes there is a small amount of sedimentation on the shoulder of the hydrometer weighing it down and causing a smaller hydrometer reading. This also would result in a coarser curve.

The application of a certain correction to the fundamental formula of Equation (3) has been proposed[11] by Mr. H. A. Lorentz, in order to take care of the increased force of friction due to frictional retardation of particles falling contiguous to the cylindrical glass wall. For hydrometer gradation purposes, however, this correction is entirely insignificant. Stokes' law gives the vertical movement of the particles; their diagonal motion, if any (especially in the lower portion of the gradation), will result in slightly higher values. Dr. Bouyoucos suggests a correction of —1 for hydrometer readings below 10 grammes per liter.

Both the elutriator and hydrometer methods are based on Stokes' law which considers particles as spherical in the derivation of the formula, and too many flat bodies in the sample will give unsatisfactory results in both cases.

The hydrometer method described herein will give reasonably accurate results, at least for the gradation of hydraulic-fill dam materials, for readily dispersible and granular soils. Undoubtedly, this method is very useful and of great assistance during periods when the results are necessary in the shortest time possible. Due to its many advantages it has become very popular and is being used in almost every soil analysis laboratory. However, the method of carrying out the test has not been standardized, as evidenced by discussions with other engineers and in the comparison of results obtained for the same soil sample in different laboratories. The tests at the Cobble Mountain Laboratory were made according to the theory and method of procedure outlined in this paper.

TURBIDITY

The standard turbidity test method can be used in estimating the percentage of a certain size of fine particles in a sample. This is only a rough approximation, and since the introduction of the aforementioned hydrometer method it has lost its value. If no hydrometer can be obtained, however, the turbidity method may be used to advantage.

Curves of particle diameter against distance below water surface are plotted from Equations (8) and (9) for various values of t. They indicate the minimum depth of various sizes of particles below the water (temperature, 10° cent.) surface after t min. of settling. In Fig. 7 the two horizontal curves give the turbidity of two different concentrations after various time intervals, t, during settling. On the right-hand side of the diagram are shown the corresponding readings of turbidity, in parts per million.

The turbidity readings will depend on various factors, such as: (1) The personal equation of the observer, especially the eyesight; (2) the intensity of the daylight, or the lack of it; shadows on the sample; (3) the shape of the particle in suspension, whether spherical or flat; (4) the color of the sample; (5) presence of organic matter in the sample; (6) temperature effect; and (7) flocculation of the material. Even with these factors, however, for the same observer and under identical conditions, there will be, according to Fig. 7, a definite relation between percentage under a certain size of fine particles and the turbidity reading of the same material.

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[11] "Colloid Chemistry", The Svedberg, 1928, p. 147.

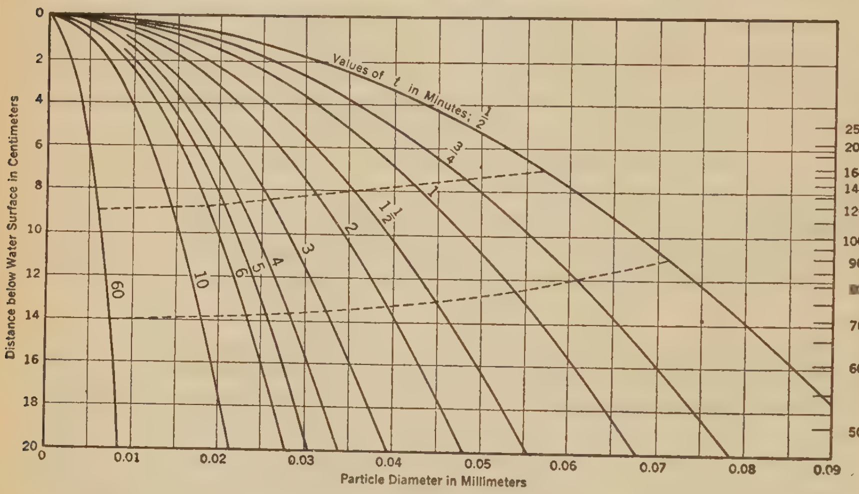

FIG. 7.—TURBIDITY CURVES SHOWING THE RELATION BETWEEN TURBIDITY AND STOKES' LAW.

Since the percentage finer than 0.01 mm. was used as a criterion, an attempt was made to find the relation between that percentage and the turbidity reading of the same material. The apparatus and the method of procedure are essentially the same as that used by the United States Geological Survey.[12]

The original concentration can be expressed in grammes of soil per liter of water. The turbidity reading is in parts per million and this is reduced to grammes per liter. This, then, divided by the original concentration, gives the percentage of original soil held in suspension at the time of observation. This ratio is called the percentage of turbidity.

A careful analysis of the same soil sample is made by a dependable method and the gradation is plotted. The percentage, from the gradation curve, finer than 0.01 mm. is divided by the percentage of turbidity, and the result is called the turbidity factor. When the percentage of turbidity and the turbidity factor of the same type of material are known, the product of the two will give the percentage of material finer than the particular size for which the factor was obtained.

Unless the material is completely broken up in the concentration while the turbidity is being taken, the turbidity reading will be very low. In some of the tests made at Cobble Mountain Reservoir of foreign samples which flocculated, it was found that the actual turbidity percentage obtained after complete dispersion of the material in suspension was as high as four times the values obtained before deflocculation. (See Table 4.)

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[12] "Measurements of Color and Turbidity in Water," *Circular 8*, Div. of Hydrography, 1902.

Readings were made 2 min. after the stirring had stopped. Various turbidity readings of the same sample were taken at intervals varying from 30 sec. to 1 hour after the stirring had stopped, and it was discovered that there was only a slight difference in the turbidity readings during the first hour of settling after the first 2 or 3 min. Evidently, the large particles, in the early stages of settling (which were above the platinum wire) had only a slight effect in reducing its visibility.

TABLE 4.—TURBIDITY FACTORS OF VARIOUS SAMPLES TESTED AT COBBLE MOUNTAIN RESERVOIR FOR PERCENTAGE OF PARTICLE SIZES FINER THAN 0.01 MILLIMETER

Location of dam	Location of samples	Number of tests made	Percentage of material finer than 0.01 millimeter, by elutriation	Percentage of turbidity	Turbidity factors
Blue Ridge, Ga	Core	1	23.0	36.1	0.64
Borden Brook, Mass	Borrow	2	8.33	17.2	0.48
Cobble Mountain, Mass	Stow borrow	60	1.89	4.16	0.45
	Russell borrow	21	11.3	18.3	0.62
	Core	103	10.56	21.25	0.50
Conklingville, N. Y	Core	3	10.45	17.10	0.61
Garza, Tex	Core	1	10.6	17.12	0.62
Rio, N. Y	Borrow	1	4.51	6.8	0.65
Rocky River, Conn	Borrow	2	9.46	17.45	0.54
Saluda, S. C	Core	1	34.0	55.7	0.61
Swinging Bridge, N. Y	Borrow	1	13.6	19.8	0.69
	Core	5	20.3	28.9	0.70
Westfield, Mass	Borrow	2	10.41	17.4	0.60
West Parish Filters, Mass	Borrow	1	7.37	12.97	0.57

ORGANIC MATTER

Organic matter is present near the surface in almost all borrow-pits, and shallow pits will have a higher percentage than deep ones. As a rule the surface soil of a dam site is sluiced, especially within the core area, and contains a high percentage of organic matter. The surface soil at the site of Cobble Mountain Dam varied from scarcely any to about 6 ft. in depth, and the average of sixteen samples showed about 8 per cent.

The determination of the percentage of organic matter is one of the important requirements. The organic matter that settles in the pool with the core material is usually very fine. It will tend to prevent the drainage of the core; it will affect the determination of the percentage of voids by moisture content and possibly the percolation test results; and it is often responsible for loss of material during elutriation tests. The effect of organic matter on computations of the percentage of voids (Equation (1)) may be expressed as follows:

$$V_t = 100\left(1 - \frac{W}{B\,s_t}\right) \qquad \text{(12)}$$

and,

$$V_o = 100 - \left(\frac{W}{B}\right)\left(\frac{a}{s_o} + \frac{100 - a}{s_i}\right) \ldots\ldots\ldots\ldots\ldots (13)$$

or, from Equations (12) and (13),

$$V_o = V_i - a\left(1 - \frac{V_i}{100}\right)\left(\frac{s_i}{s_o} - 1\right) \ldots\ldots\ldots\ldots\ldots (14)$$

in which, V_o is the percentage of voids observing effect of organic matter; V_i, percentage of voids neglecting effect of organic matter; s_o, specific gravity of organic matter; s_i, specific gravity of soil particles; a, percentage of organic matter by weight; W, weight of sample, in grammes; and B, volume occupied by sample, in cubic centimeters. The last term on the right-hand side of Equation (14) is a correction because of organic matter to be applied to the computations of percentage of voids, V_i, as ordinarily computed.

Assuming that $s_i = 2.72$, and $s_o = 0.60$, Equation (14) becomes:

$$V_o = V_i - 3.53\, a \left(1 - \frac{V_i}{100}\right) \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

It is interesting to study the relation between V_o and V_i for various percentages of organic matter, according to Equation (15). For instance, for a 3% organic matter ($a = 3$), with the computed percentage of voids, $V_i = 53$, it is seen that the actual percentage of voids is only 48. In other words, 3% of organic matter in the core material makes a difference of 5% in the computations of the percentage of voids of the material, had the organic matter been neglected. Equation (15) will apply for any other values of s_i and s_o and for their different values curves can be plotted to facilitate computation.

Organic matter can be determined by the method of "loss on ignition." This method is not reliable, unless the test is run carefully because minerals may decompose during the test if the sample is overheated, thus giving higher results for organic matter.

The albuminoid ammonia method was used for a few determinations of organic content, but was discarded later since reasonably accurate results could be obtained more easily and in a shorter time by igniting a sample.

Colorimetric tests for the determination of organic matter, similar to those made for organic impurities in concrete sands, were tried. Due to the fact that the cores of hydraulic-fill dams have a higher percentage of organic matter than concrete sands, a smaller ratio of sample material was used. It may be possible to shorten and simplify this method so that it can be used to advantage in the determination of organic content of materials for hydraulic-fill dams.

GRADATION OF CORE MATERIALS

With due respect to the difference in opinions regarding the many factors affecting the suitability of the core material other than its size gradation, the writer believes that one of the most important criterions to decide this suitability is the grain size and the gradation of the core material.

There has been much controversy among engineers in regard to the proper sizes of the core material for hydraulic-fill dams. Every engineer who has built a successful dam is inclined to insist that the sizes of his core are the proper ones to be used. Unfortunately, every engineer has his own method of determining gradation, and the question arises as to whether the different results give a fair basis of comparison.

Numerous samples of core and borrow materials from important hydraulic-fill dams were secured and tested at the Cobble Mountain Reservoir with the same method and under the same conditions, so as to make the results comparable. In Fig. 8 the numbers in parentheses denote the number of samples tested and averaged in each case for the resulting curve. The Alexander Dam, at Kauai Island, Hawaii, collapsed in 1930, during construction. The gradation shown for this dam is that of the material before failure. The heights of these dams are: Blue Ridge, Ga., 170 ft.; Calaveras, Calif., 225 ft.; Cobble Mountain Dam, Mass., 263 ft.; Conklingville, N. Y., 95 ft.; Garza, Tex., 80 ft.; Gatun Dam, Canal Zone, 115 ft.; Germantown, Ohio, 110 ft.; Cambria Steel Company Dam, Johnstown, Pa., 95 ft.; City Dam, Johnstown, Pa., 70 ft.; Rocky River Dam, New Milford, Conn., 92 ft.; Saluda, S. C., 208 ft.; Sherman Dam, Monroe, Mass., 100 ft.; Somerset, Vt., 110 ft.; Swinging Bridge Dam, Forestburg, N. Y., 135 ft.; and Wichita Falls, Tex., 100 ft.

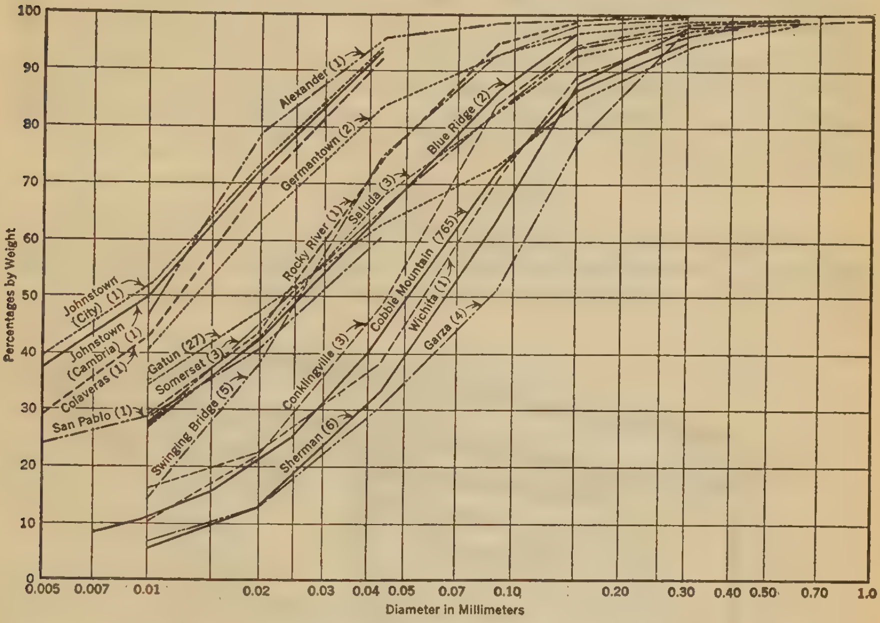

FIG. 8.—SUMMARY OF GRADING ANALYSES, MADE AT COBBLE MOUNTAIN RESERVOIR OF CORE SAMPLES, FROM VARIOUS HYDRAULIC-FILL DAMS.

Scarcely, if ever, will the core material of a hydraulic-fill dam have the same grain sizes and gradation throughout the body of the core. One sample, or a few samples, of any core will not necessarily correspond to a representative average of the entire core material. It is necessary, therefore, to make frequent and numerous gradation analyses in order to obtain a fair average gradation curve for the entire core of a dam. The writer doubts whether the gradations given in the summary sheet (Fig. 8) will really represent the average of the entire core of any dam represented by the analysis of any one of its samples. Unfortunately, it was impossible to secure the desired number of samples for every dam. Otherwise, the summary sheet, Fig. 8, is believed to be of real value for a comparison between various core material gradations, since no error is present due to difference of method and to personal equations. It is well to call attention here to the fact that each engineer plots gradation curves differently. The gradation plotted in the first quadrant, with "diameter sizes, in millimeters" increasing from left to right as abscissas and "percentage of material finer by weight" increasing from the bottom upward as ordinates, seems to be the most logical.

Coefficient of Friction

The beach materials of hydraulic-fill dams consist generally of clean sand and gravel. The coefficient of friction of these materials is of great importance in design, but, unfortunately, no reliable data are available.

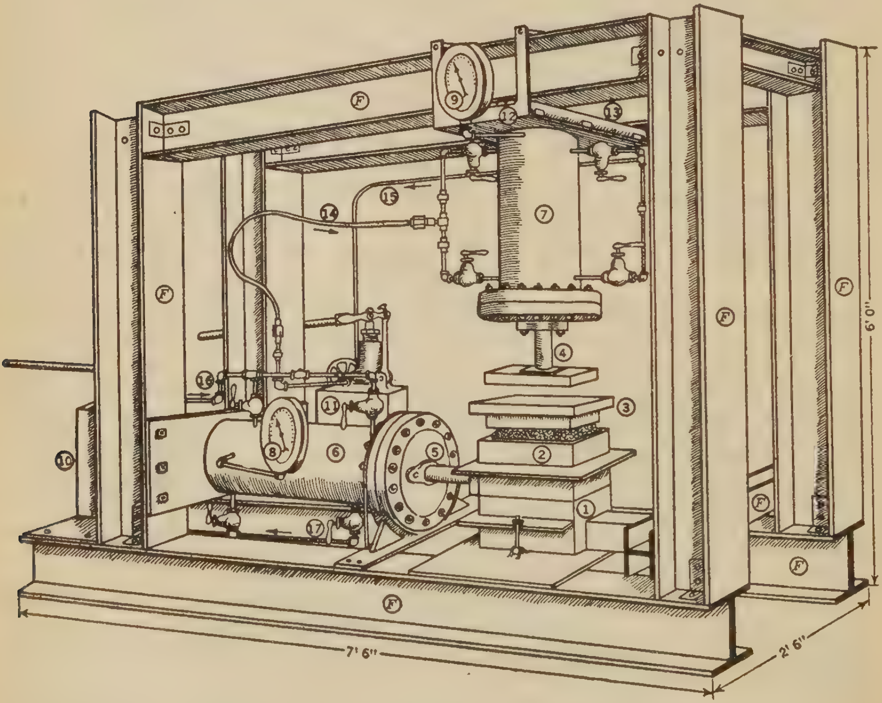

Fig. 9.—Apparatus for Determining Coefficients of Friction.

The apparatus used for measuring the coefficient of friction of materials at Cobble Mountain (see Fig. 9) was specially designed for the purpose. It consists of a rigid steel frame, *F*, electrically welded throughout, weighing about 2 tons. Within this frame the essential working parts (labeled in Fig. 9) are, as follows: Vertical pressure is applied to the material within the sample boxes (1) and (2), by means of the loading block (3), which, in turn, gets its force from the piston shaft (4), operated by water under pressure within the cylinder (7). This hydraulic pressure is obtained by a hydraulic hand-pump (11), by way of the feed-pipe (14). The horizontal force required to shear the sample is applied to the upper box (2), in a similar manner by pump (10), feed-pipe (16), horizontal cylinder (6), and piston shaft (5). Hydraulic pressure in both the horizontal and vertical cylinders is recorded on 1 500-lb. pressure gauges (8) and (9), respectively. The vertical cylinder is mounted on a carriage (12), holding rollers (13), to permit horizontal movement as shear takes place. As the piston within the cylinders moves in either direction, water is forced back to the pumps through return pipes (15) and (17) for further use.

The procedure for determining coefficients of friction is:

(*a*) Weigh out about 100 lb. (dry weight) of sample and place enough of it in the lower sample box (1), agitating the material in the box under water for complete saturation.

(*b*) Place the upper half of the sample box (2) into position and continue placing the sample material to the top of the box in the same manner as in Step (*a*).

(*c*) Allow the sample material to drain partly for 5 to 10 min. Place the loading platform (3), leaving sufficient clearance between the platform and the sides of the sample box.

(*d*) Take measurements of the height of the sample in the box and record.

(*e*) Apply the vertical load (4) gradually on the sample, to pressure (9) desired for test.

(*f*) Allow the sample material to consolidate for 5 to 15 min., keeping the pressure over it constant by a slight pressure applied to the handle of the pump (mentioned in Step (*e*)).

(*g*) Release the vertical pressure, jack the two sample boxes apart about $\frac{1}{4}$ in., and leave them blocked apart.

(*h*) Apply vertical pressure as in Step (*e*) and allow the material to consolidate further.

(*i*) Remove the blocks between the sample boxes.

(*j*) Measure the height of the compressed sample material (for computing the consolidation) and record.

(*k*) Apply pressure slowly by means of the horizontal piston (5) to the upper half of the sample box until there is no increase in the pressure observed on the corresponding gauge (8); any additional horizontal pressure at this point would cause the upper box to slide which would indicate that the sample has sheared.

(*l*) Observe both the horizontal (8) and vertical (9) gauge readings at this point, Step (*k*), and record.

(*m*) From the foregoing records compute:

$$L = \frac{G A_p}{A_s} \qquad (16)$$

in which, L is the vertical pressure, in pounds per square inch; G, the vertical gauge reading; A_p, the area of the piston; and A_s, the area of the sample. Next compute the coefficient of friction from the relation: Coefficient of friction $= \frac{\text{horizontal gauge reading}}{\text{vertical gauge reading}}$. Finally, the percentage of voids is determined from,

$$V = \frac{100}{B}\left(B - \frac{W}{s}\right) \qquad (17)$$

in which, B is the measured volume of sample; s, the specific gravity of material; and W, the dry weight of sample.

(*n*) Release the pressures and remove the sample.

(*o*) Make independent and complete coefficient-of-friction tests, repeating Steps (*a*) to (*n*) for various desired vertical pressures up to 400 or 500 lb. per sq. in.

(*p*) Plot the results with the coefficient of friction as ordinate, and vertical pressure, in pounds per square inch, as abscissa.

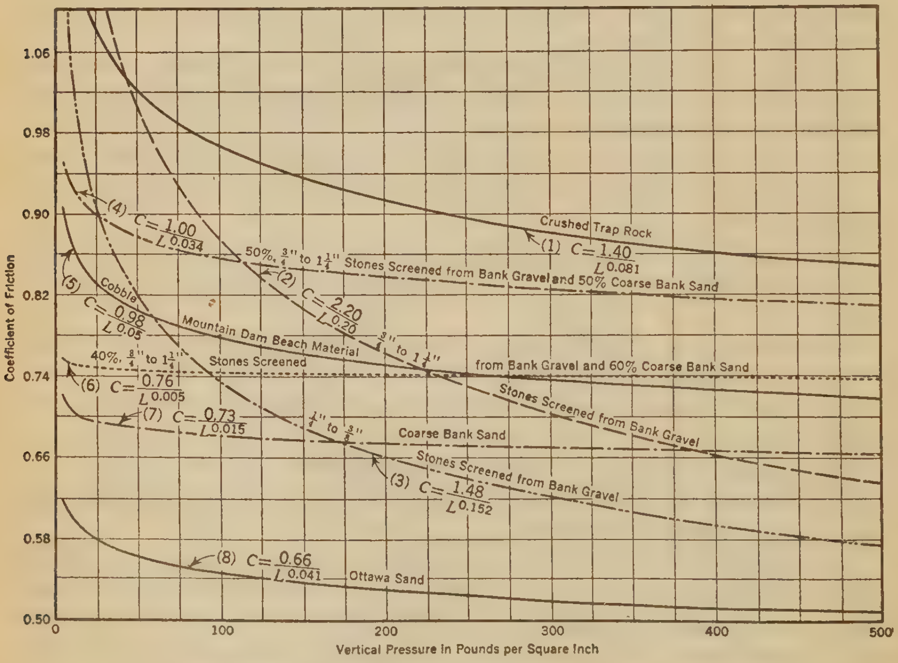

FIG. 10.—COEFFICIENT-OF-FRICTION CURVES FOR VARIOUS MATERIALS.

Curves resulting from about 550 coefficient-of-friction tests made at Cobble Mountain Reservoir, are given on Fig. 10. It will be seen that the coefficient is not constant, but is expressed by the general formula:

$$C = \frac{k}{L^m} \quad \text{.............................} (18)$$

in which, C is the coefficient of friction; k, the constant; L, the vertical pressure in pounds per square inch; and m, the exponent of L.

It is further seen that k, the constant for any one material, varies from 0.66 to 2.20, and that values of m, the exponent of L, range from 0.005 to 0.20, depending on the characteristics of the material tested.

Due to the fact that the beach materials of a hydraulic-fill dam are placed by water and that after construction they are never absolutely dry, the test materials were placed by water in the sample boxes, and, after draining all free water, the tests were conducted while the material was wet. The data given in Fig. 10 are based on the foregoing conditions.

Although primarily interested in the coefficient-of-friction values of the beach and core materials of Cobble Mountain Dam, more than 200 tests were made on outside materials consisting of various local and foreign samples of sand, gravel, crushed trap-rock and screenings, and Ottawa sand. The coefficients for the materials in question are affected by their particle sizes, hardness, percentage of voids, moisture content, particle shape, and gradation.

The conclusion drawn from the tests at Cobble Mountain Reservoir is that, other factors remaining the same, for any given material, higher values of the coefficient of friction are obtained by: (1) Larger particle sizes; (2) harder material; (3) smaller percentage of voids; (4) dryer material; (5) particles of more angular shapes; and (6) better graded material, or the combination of Factors (1) and (3).

The tests showed conclusively that larger particle sizes gave higher coefficients of friction. The interlocking effect of stones seems to be an important factor. In the rock toes of Cobble Mountain Dam where the maximum particle sizes with irregular shapes are about 20 cu. yd., it is difficult to estimate the increase in the coefficient-of-friction value.

The hardness of material is important. No material was tested, that did not show the effect of crumbling or breaking after shearing under vertical pressure. Softer materials crumbled more, thus causing considerable decrease in particle sizes; the larger sizes crushed more than the smaller ones, and irregular-shaped particles more than the rounded ones.

The curves in Fig. 11 give the amount of crumbling for tests on different materials at various pressures. The results are from gradings made on samples taken at the shearing plane before and after a test. The increase in percentages plotted are based on the finest sieve of the size range shown for each material, except for the coarse bank sand and the crushed trap-rock screenings, for which No. 28 (0.62-mm.) and No. 14 (1.27-mm.) sieves were used, respectively. A new sample of the original material was used for each pressure. In every case, it is seen that the crushing effect increases with an increase in pressure, thus reducing the particle sizes. Undoubtedly, this is greatly, if not entirely, responsible for the decrease in the coefficient-of-friction values for higher pressures (Fig. 10).

For the same pressure a given material with a smaller percentage of voids yielded a higher value for the coefficient of friction. However, there is a limit to minimum percentage of voids because, under a given vertical pressure, the percentage of voids of a material will eventually become almost constant. Slight variation in the consolidation under the same pressure may be due to the final arrangement or self-placement of the particles, or, perhaps, to the crumbling of particles to a certain extent.

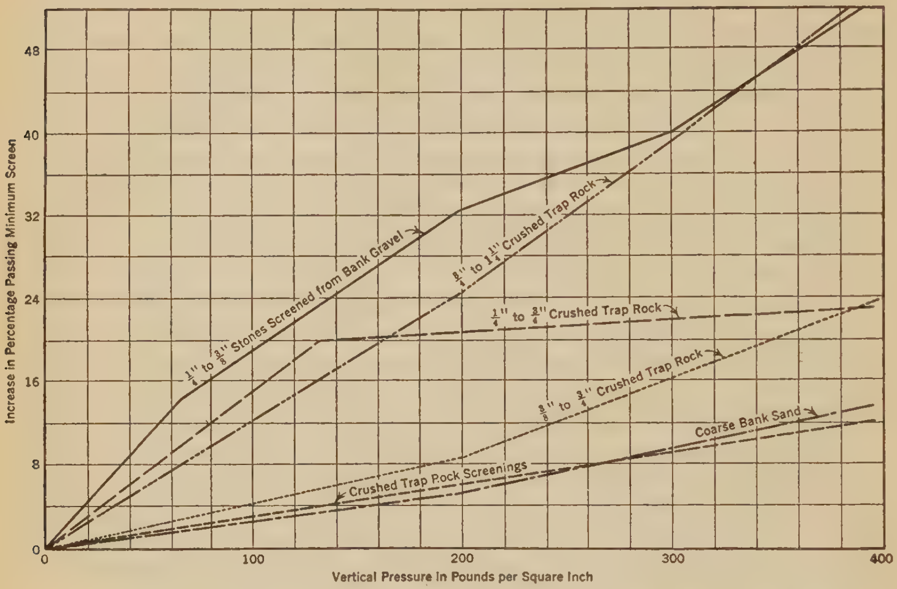

FIG. 11.—CURVES SHOWING AMOUNT THAT MATERIAL CRUSHED DURING TESTS FOR COEFFICIENT OF FRICTION.

When the particles of the material carrying the load are wet, the moisture seems to act as a lubricant, and there is a decided decrease in the value of the coefficients, especially at low pressure. The same material dried and tested for coefficient of friction under a given vertical load gives higher values. In Fig. 12, which illustrates this point, the increase in the coefficient of friction is based on the coefficient of the material in a wet condition; that is, the surfaces of all particles are entirely covered with water.

The more irregular the shape of particles the more pronounced is the effect of interlocking, resulting in increased resistance for friction. The easy crumbling of the sharp edges of particles with angular shapes may offset this interlocking advantage somewhat. Unless the material is hard, the sharp corners may not prove advantageous. The more uniform the particle sizes the higher is the percentage of voids. Well-graded beach material containing coarse particles has smaller percentages of voids and will have higher friction values.

The foregoing data are by no means all that are desirable as to the coefficient of friction of materials in hydraulic-fill dams. Unfortunately, at Cobble Mountain Reservoir it was not possible to make further investigations along this line. In this large field much more reliable information is necessary to determine the characteristics of various materials, in order to understand fully the behavior of the coefficient of friction.

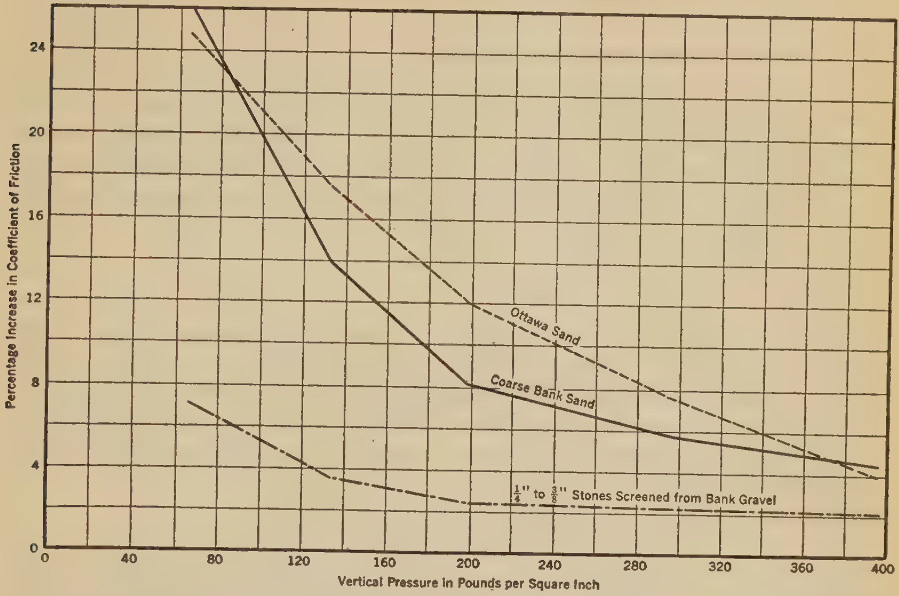

FIG. 12.—CURVES SHOWING INCREASE IN THE COEFFICIENT OF FRICTION DUE TO CHANGE FROM WET TO DRY CONDITION.

PERCOLATION

The percolation, or the permeability, test is one of the practical criterions of the ultimate behavior of the dam; and, therefore, this test is important and should be carried on with the greatest care.

The purpose of the gradation, or the determination, of the grain size of the core material is to ascertain whether the material has enough fines so that it will be impervious, or reasonably so, and whether it is coarse enough so that it will drain and solidify within the near future. As will be seen elsewhere in this paper, the effective size of the material, obtained from the gradation curve, will be useful in estimating its coefficient of permeability. It is better, however, to determine the coefficient of permeability directly rather than rely on gradation alone, as seepage through a core material will depend not only on the effective size and gradation of its materials, but also on its percentage of voids, its percentage of organic matter, its resistance to consolidation, or the shape and toughness of the particles, and on its percentage of clay, especially within the colloidal ranges, etc.

The velocity of seepage through fine core material is so slow that units, such as centimeters per second, or feet per second, will result in small quantities. It is advisable to express this velocity in meters per day, which will result in numbers that are more convenient. The coefficient of permeability, K, in meters per day, is also equal to cd^2, in which c is a constant and d is the effective size of the material.

As the normal ground-water temperature is about 10° cent., and as the change in temperature will affect the velocity, it is better to base the coefficient of permeability on 10° cent., or 50° Fahr. In dealing with the determination of the values of K, 45% voids consolidation was used as a basis. The fundamental formula used in determining the velocity of seepage through soils is known as Darcy's law:[13]

$$v = K \frac{H}{l_p} \quad \dots\dots\dots\dots\dots\dots\dots\dots (19)$$

in which, v is the velocity of flow; K, the coefficient of permeability; H, the head causing the flow; and l_p, the length of the path of percolation. Since H and l_p are known, or can be assumed, the value of K is required. Due to the fact that the experiments to determine the value of K require long periods of time and constant watching, attempts have been made to develop short-cut methods.

STAND-PIPE PERMEAMETER

Both the vertical capillary rise[14] and the horizontal capillary flow[15] methods of measuring permeability of soil were tried at Cobble Mountain Dam and were found unsuitable for core materials in a hydraulic-fill dam.

The simplest method of measuring the permeability of a soil is by a stand-pipe permeameter. In the apparatus used at Cobble Mountain shown in Fig. 13 the core material, in a saturated state similar to that in the dam, is placed in a glass tube and immersed in a pail of water. The head causing flow is measured from the top of the column of water in the stand-pipe to the level of the water in the pail. By means of a stop-watch the rate of fall of the water column in the stand-pipe was observed, and its volume was obtained when the bore of the stand-pipe had been determined. From the moisture content of the sample its percentage of voids is computed.

The coefficient of permeability, K, in meters per day, is computed from the equation:

$$K = 1\,989.44 \frac{A\, l_p}{A_s\,(T_2 - T_1)} \log \frac{H_1}{H_2} \quad \dots\dots\dots\dots\dots (20)$$

in which, A is the cross-section area of the stand-pipe; A_s, the cross-section area of the sample; l_p, the thickness of the sample; T, time; and H, the head that causes flow; that is, H equals height of water in the stand-pipe, H_1 at the beginning, and H_2, at the end, of the run. Equation (20) is derived from Darcy's law and is reduced from the observed temperature to its value at 10° cent. by means of Fig. 5. During any of the tests—for the same material, with the same percentage of voids, and whatever the value of H_1 in Equation (20)—K remained constant, provided the sample maintained structural equilibrium during the runs.

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[13] "Rechérches Expérimentales Relatives au Mouvement de l'Eau dans les Tuyaux," par. Henry Darcy, Paris, 1857.

[14] "Soil Studies for the Granville Dam in Westfield, Mass.", by Charles Terzaghi, M. Am. Soc. C. E., *Journal*, New England Water Works Assoc., June, 1929, Vol. 43, No. 2, p. 191.

[15] "Soil Mechanics Research," by Glennon Gilboy, Jun. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 98 (1934), p. 225, Equation (5).

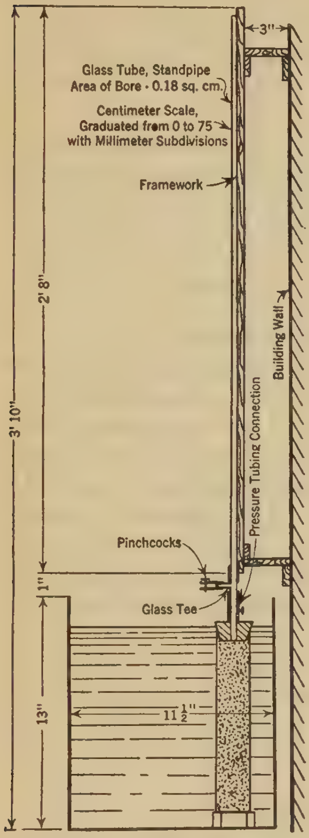
3"
Glass Tube, Standpipe Area of Bore = 0.18 sq. cm.
Centimeter Scale, Graduated from 0 to 75 with Millimeter Subdivisions
Framework
2' 8"
3' 10"
Building Wall
Pressure Tubing Connection
Pinchcocks
1"
Glass Tee
13"
11 1/2"

FIG. 13.—STAND-PIPE PERMEAMETER.

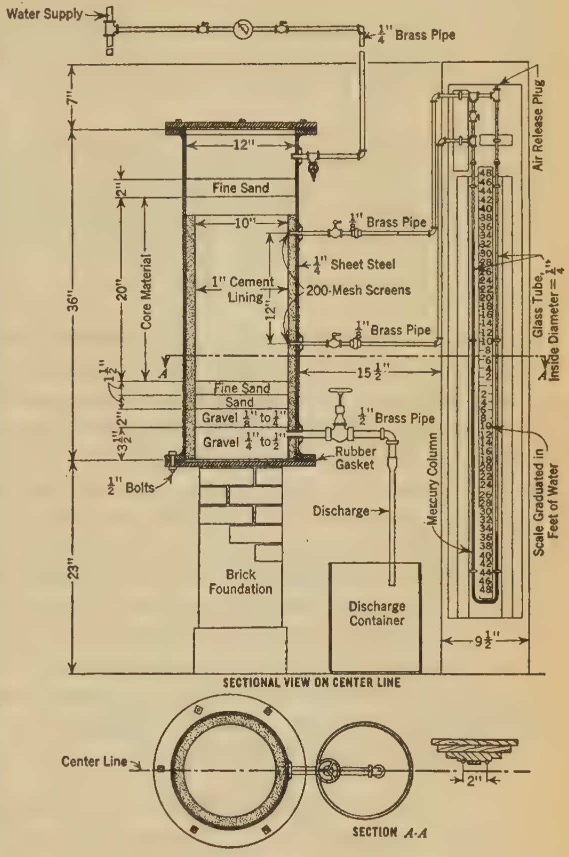
Water Supply
1/4" Brass Pipe
Air Release Plug
7"
12"
Fine Sand
2"
10"
1/8" Brass Pipe
1/4" Sheet Steel
1" Cement Lining
200-Mesh Screens
12"
20"
Core Material
36"
1/8" Brass Pipe
Glass Tube, Inside Diameter = 1/4"
A
15 1/2"
1 1/2"
Fine Sand
Sand
Gravel 1/8" to 1/4"
Gravel 1/4" to 1/2"
2"
3 1/2"
1/2" Brass Pipe
Rubber Gasket
1/2" Bolts
Mercury Column
Scale Graduated in Feet of Water
Discharge
23"
Brick Foundation
Discharge Container
9 1/2"
SECTIONAL VIEW ON CENTER LINE
Center Line
2"
SECTION A-A

FIG. 14.—LARGE PERCOLATOR.

It should be remembered that the stand-pipe permeameter has a low head and the sample to be tested is very small. Furthermore, there is a chance for the water to flow more freely contiguous to the inside wall of the glass cylinder.

LARGE PERCOLATOR

This apparatus (Fig. 14), designed by the late Allen Hazen, M. Am. Soc. C. E., can be used with a high head of water pressure. It has a capacity for core material of about 1 cu. ft., which is sufficient to reduce the effects of slight local variations in the sample. It was lined with concrete forming a collar near the top of the sample to check seepage contiguous to the inside surface (see Fig. 14).

The sample is placed in a saturated state as it comes from the core, and is supported and covered by sand filters. After a period of initial settlement (about 24 hours), the container is sealed and the water pressure is applied at the top of the sample in gradually increasing amounts, to the desired limit. A maximum head of water of about 60 ft., was used at Cobble Mountain Reservoir.

In order to avoid the effect of a possible film of fines at the surface of the sample, the head was measured by a differential mercury manometer connected at points, 1 ft. apart, lying within the sample. The readings gave a direct measurement of the head causing the flow through 1 ft. of the material, or the value of $\frac{H}{l_p}$ in Equation (19). The discharge was measured at convenient intervals, when the temperature was observed.

The values of K in Equation (19) decreased rapidly during the first few hours of the test, and continued to decrease slowly for a long time. The initial tests were carried on over a period of from two to three months, when the coefficient of permeability became nearly constant. It was found, however, that satisfactory comparable tests could be terminated in a week. The initial rapid decrease was due to the fact that the applied pressure produced a decrease in the percentage of voids which was accompanied by settling of the sample. All values of K were reduced to 10° cent. for comparison. The voids of the core sample were found at the beginning and at the end of the test, by moisture-content determinations.

The large percolator is satisfactory for the determination of the coefficient of permeability for a soil sample at a single percentage of voids, but it is not suitable for the measurement of consolidation, or the corresponding variations in the coefficient of permeability.

STANDARD COMPRESSION APPARATUS

The best device for the determination of the relation between K and the percentage of voids, is that by Professor Terzaghi, described by Professor Gilboy in the paper previously cited.[15] This device (called the standard compression apparatus) will measure not only the rate of consolidation and compressibility of a soil sample under a given load, but also its permeability at different stages of consolidation. No description of the apparatus is necessary here, as it has been described often and is fairly well known. The loading system is made in such a way that the compressibility of core material can be measured with a load of 200 lb. per sq. in., which is about the maximum pressure in the Cobble Mountain Dam.

After the sample is placed in the machine, the stop-pin is removed and consolidation is allowed to take place under the weight of the piston. When this is practically complete, the stand-pipe connection is made. Increments of weight are applied in order to give the desired pressures up to 200 lb.

per sq. in. At each loading observations are made to determine the rate of consolidation under that load and sufficient time is allowed for complete consolidation. The permeability after complete consolidation is observed for each loading.

The increments of weights are removed in the order that they were placed, and sufficient time is allowed for complete expansion between the removal of each load. Additional observations may be made for permeability at this time.

The average of the readings of two Ames dials is used for computing the consolidation. The coefficient of permeability is computed from Equation (20) and reduced to its equivalent value at 10° cent. The following data are obtained for a given material: (*a*) The rate of consolidation for each loading; (*b*) the compressibility and expansion; and (*c*) the coefficient of permeability.

Numerous tests were run with the standard compression unit and valuable data obtained. It was unfortunate, however, to note that the coefficient of permeability of the same sample and for the same percentage of voids obtained by the standard compression apparatus was invariably lower than, and in some cases one-half, the results obtained from the large percolator. The variation of the results, for each apparatus and for different percentages of voids, was consistent. There was no apparent reason to doubt the results obtained by the large percolator because no difference was found in the value of K for different values of H. The sample of the standard compression apparatus is so small that a personal equation or any slight cause may result in a great difference in the result of the test. The following might account for the small values of K obtained by the standard compression apparatus: (*a*) The inside area of the piston bearing on the top porous plate is 28.7 sq. cm., whereas the area of the sample is 38.1 sq. cm.; thus, the water of about 25% of the sample area is forced to come in a diagonal direction; (*b*) there is a possibility that the top porous plate may become clogged to a certain extent; and (*c*) the puddling of the sample in placing it in the cylinder may cause segregation.

The results obtained for the coefficient of permeability of fine core materials are so small and subject to so many different factors that even, say, about 100% variation between two different apparatus cannot be considered serious.

PERCOLATION AND PERCENTAGE OF VOIDS

In the numerous experiments for the coefficient of permeability of any one material, K remained constant for any value of H, provided the percentage of voids of the material remained constant and the sample maintained structural equilibrium, but it showed a decided change due to variation in the percentage of voids.

The curves of Fig. 15 show the relation between the value of K and the percentage of voids of several core samples. This relation can be expressed by the formula,

$$K = c\,p^{m} \qquad (21)$$

in which, K is the coefficient of permeability, in meters per day, and for $t = 10°$ cent., c, is a constant, and p, the percentage of voids divided by 100. The value of c varied from 2.86 to 0.48 and the value of m, from 5.40 to 7.20.

For the same material, it can be stated that K varies approximately as the sixth power of the percentage of voids. Thus, the value of K for 50% voids compared with that of 40% will be about $\left(\frac{50}{40}\right)^6$, or 3.8 times. This relation, derived from tests on local core samples, will be reasonably accurate for local material. The relation might be similar for other core materials, provided their grading and character are similar.

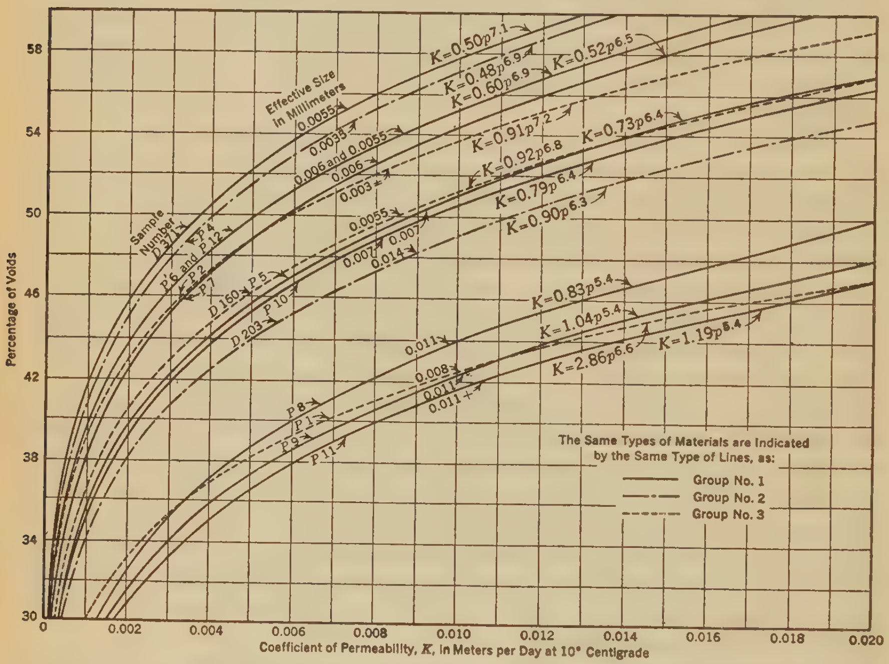

FIG. 15.—PERCENTAGE OF VOIDS PLOTTED AGAINST COEFFICIENTS OF PERMEABILITY FOR VARIOUS MATERIALS AND EFFECTIVE SIZES.

In Fig. 15, the values of K point out three distinct characteristics; namely, that they (1) differ considerably for different materials; (2) vary most decidedly according to the effective size of each sample of the same character of material; and (3) vary according to the slope of the gradation curves of the same character of material even if the materials may have the same effective size.

Fig. 16 gives the gradations of the materials of the curves of Fig. 15. Comparing $P5$ and $P10$, both being the same kind of material and having the same effective size—0.007 mm.—and the value of m being 6.4 in both cases, the value of c for $P5$ is 0.73, and for $P10$, it is 0.79. On Fig. 16, it is seen that $P5$ is finer than $P10$ with a difference of about 0.005 mm. in their grain sizes for the entire range between the 35% size and the 60% size, or the $P10$ grain sizes are 16% larger than $P5$ in that range. This causes about 8% variation in the rate of percolation of the two samples.

A similar comparison of $D371$ with $P12$ shows that the latter has a coefficient of permeability about 40% greater than the former. From Fig. 16 it is seen that the curve of $P12$ is coarser from the 20% to the 60% size. The particles in this range have an average difference in diameter of about 17 per cent. Furthermore, for the same range the particle sizes of $P11$ are about 27% greater than those of $P8$, while $P11$ is about 43% more permeable than $P8$. This, in spite of the fact that the effective sizes of the two samples, in both the foregoing cases are the same.

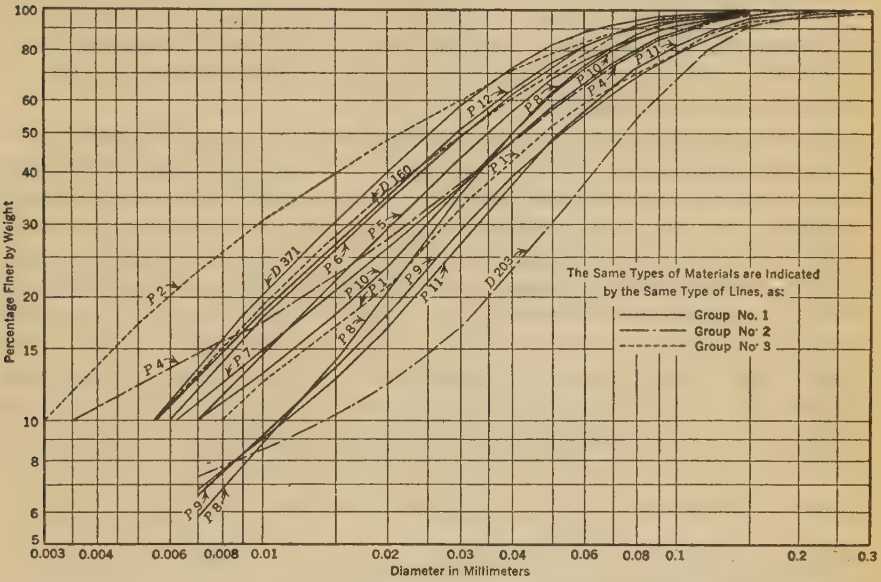

Fig. 16.—Gradation of Samples Tested for Determination of Relation Between Coefficient of Permeability and Percentage of Voids, as Shown in Fig. 15.

Effective Size

Many claim that effective size is an arbitrary term, stating that it may suffice for filter sand where the gradation is more or less uniform and doubt its application to a core material. It is true that without the uniformity coefficient the effective size of a material will give no idea in regard to its gradation. A great many curves can have the same effective size with enormous difference in the gradation of the materials. It should be remembered, however, that the minimum and maximum 60% sizes of the core materials (Fig. 8) are about 0.013 mm. and 0.11 mm., respectively, and that their average value is about 0.035 mm. The effective size term, when applied to a core material, therefore, gives a fair notion of the gradation of that material, especially when the term definitely fixes in mind the maximum size of the 10% fines in the total core.

Mr. Hazen's formula[16] for filter sands is:

$$v = c\,d^2 \frac{H}{l_p} \left(\frac{t + 10}{60} \right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

It is taken from Darcy's law as expressed by Equation (19), in which, v is the velocity of water, in meters per day; c, a coefficient; d, the effective size, in millimeters; H, the head causing the flow; l_p, the depth of material (filter sand in Hazen's formula); t, the temperature, in degrees Fahrenheit; and K, the coefficient of permeability.

For $t = 50°$ Fahr., or 10° cent. (which is the normal temperature for ground-water) Hazen's $c\,d^2$ becomes equal to Darcy's K. Numerous tests were run to determine the coefficient of permeability, K, for the core material of Cobble Mountain Dam, and, at the same time, to find out whether $c\,d^2$ would be applicable to flows through the extremely fine core materials. The effective sizes of these materials subject to experiments varied from 0.003 mm. to 0.012 mm., and it was discovered that at 45% voids the value of c remained reasonably constant, with a mean average of about 150. The conclusion was reached that Hazen's factor, $c\,d^2$, can be applied approximately for seepage through these fine core materials with $c = 150$. It should be remembered that in using $K = c\,d^2 = 150\,d^2$, K is the coefficient of permeability for velocity, in meters per day, for $t = 50°$ Fahr., and at 45% voids. For any other temperature, the factor, $\frac{t + 10}{60}$, of Equation (22) should be applied and for different percentages of voids Equation (21) can be applied; or perhaps it can be better expressed by:

$$K = 150\,d^2 \left(\frac{V}{45} \right)^6 \left(\frac{t + 10}{60} \right) \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

in which, K is the coefficient of permeability, in meters per day; d, effective size, in millimeters; V, percentage of voids; and t, temperature, in degrees Fahrenheit.

The value of c may vary considerably from 150, due to variations in the character and gradation of different core materials. For a given locality, however, where the materials are of a similar character and grading, c can be evaluated and used with reasonable accuracy.

It is better, of course, to conduct tests to measure directly the coefficient of permeability of the core material of a dam under construction, in order to calculate, more or less accurately, the probable seepage through the dam, but Equation (23) is a handy tool for computing at least approximately, the probable seepage through the dam before its construction. This might result in very effective changes in the design of the contemplated structure, or in locating other than the anticipated borrow-pits to be used for the construction.

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[16] Annual Rept., Massachusetts State Board of Health, 1892, p. 539.

The significance of effective size is appreciated and its importance is realized due to the fact that the seepage, all other factors influencing it remaining the same, varies as the square of the effective size. Thus, a computed seepage through a dam in which the material has an effective size of 0.008 mm. will be increased nine times if the effective size is changed to 0.024 mm.

It is hoped that in the future, with more comprehensive investigations, a factor may be obtained for Equation (23) to take into consideration the effect of the slope of the gradation curve. (See Characteristic (3) under "Percolation and Percentage of Voids.")

Effective Size Estimated by Observation

It is surprising to know how easily an inspector at the dam can be trained to judge the approximate effective size of the core material by visual examination and touch. The method of comparing the material from the dam with a sample of similar material that has been tested, is used to train the inspector, and to maintain a constant field check on the work. A series of samples having effective sizes from 0.002 to 0.02 mm. were kept in the field for this purpose.

It was discovered that a core material with an effective size of about 0.002 mm., when rubbed between the fingers, felt like graphite; it was greasy like butter. An experienced inspector can tell the character of a core material in regard to its fineness by observing the flow while it is going over the beach. These observations may not apply directly to other dams, but similar observations may be found useful.

Consolidation of the Core of Cobble Mountain Dam

An attempt has been made to estimate roughly the time necessary for an approximate ultimate consolidation of the core at various heights of the Cobble Mountain Dam. By "approximate consolidation" for various heights of the dam is meant the state of consolidation that the material at any height will attain due to the pressure or weight of the structure itself above that point.

In the laboratory, numerous samples of core material were tested in the standard compression apparatus and corresponding consolidation curves were obtained. For a given material, these curves showed the ultimate percentage of voids of the material under a given load. The application of pressure was made in increments from a very small load up to 200 lb. per sq. in.

It was discovered that the ultimate percentage of voids was affected by the arrangement of the particles (the percentage of voids) at the beginning of the tests and by the effective size of the material. The lower the percentage of voids at the beginning of a test, the lower will be the final percentage of voids. With materials of similar gradation, the larger the effective size, the lower will be the percentage of voids throughout the test.

The average of the laboratory results for local core material with an effective size of 0.008 mm., can be expressed by the equation:

$$100 - V = 49.3\,L^{0.0428} \quad \dots\dots\dots\dots\dots\dots\dots (24)$$

in which, V is the final percentage of voids (expressed in whole numbers, thus, $45\% = 45$ and not 0.45); and L is the pressure, in pounds per square inch, causing ultimate consolidation to $V\%$ voids.

It is questionable whether the consolidation formula obtained in the laboratory will apply to the actual consolidation of the same material in the construction of the dam because of the following reasons:

1.—The applications of the loads in the laboratory and in the dam are different.

2.—The initial percentage of voids or the arrangement of the particles in the testing machine and in the dam is not the same.

3.—The sample for the test in the standard compression machine is very small. It was found that there is a great difference in the time necessary for the ultimate consolidation during a given period under a given loading.

4.—There is a great difference between the inside polished surface of the standard compression unit of the apparatus and the rough surfaces of the abutments and the beaches in the dam.

5.—The time interval for a given load is different.

6.—The drainage facilities are not the same.

7.—There is the question of the load per square inch at any one given depth in the dam, due to the uncertainty of the behavior of the core material in coming to a state of structural equilibrium, or in the rate of consolidation taking place in the upper part of the dam above that particular point. As the percentage of voids in the core decreases, the weight of the core material per cubic foot increases, thus causing a higher pressure per unit area at any depth of the dam.

The consolidation data obtained for Birch Meadow borrow materials may not necessarily apply to other borrow materials in the dam. The results and the formula obtained, should be used with discretion. The application of the formula may not be entirely correct with respect to the actual conditions in the dam; but at present it is the best available information, and, undoubtedly, it is a tool that can be utilized to compute approximately the time of consolidation at various depths.

At the completion of the dam, if the percentage of voids of the core material at any height can be assumed, and the final percentage of voids ascertained from the consolidation formula at that height, the quantity of water to go through a vertical area at the same elevation in the dam can be calculated. The width of the core at any height will determine the path of percolation of the water to be squeezed out at that height; hence, the total quantity of water that will have to drain away can also be determined. Using a certain average velocity of the water, the time necessary for it to drain away can be computed.

It is evident that, in addition to the uncertainties to the application of the consolidation formula, in order to solve the problem, other assumptions are being made, which perhaps are not exactly right.

For example, let B equal the volume of water, in cubic meters, to drain from the core material with a given percentage of voids until it attains the final percentage of voids under a given load; Q, the discharge, in cubic meters per day; v, the average velocity, in meters per day, for V and V'; A, the vertical area, in square meters, perpendicular to direction of flow; T, the number of days; H, the height, in meters, or in feet, below the top of the dam; D, the width of core, in meters, or in feet, at H distance below the top of the dam; V, the initial percentage of voids of core material; V', the final percentage of voids of core material; and d, the effective size of core material, in millimeters.

Assume that horizontal and longitudinal drainage of the water occurs in the core material during consolidation. (This is not exactly the case, as there may always be vertical and diagonal drainage.) Assume, also, that the reservoir is full and that the water in the core material during consolidation cannot flow up stream and that it will drain the full width of the core horizontally down stream. Then, $B = QT = AvT$, or,

$$T = \frac{B}{Av} \qquad \text{(25)}$$

The volume of water, B, to be drained within a section of the core during the change of percentage of voids from V to V' is equal to the original minus the final water content of the section.

With a percentage of voids of V, there are $\frac{V}{100} DA$ units of water and $\frac{100-V}{100} DA$ units of solid material. When the percentage of voids becomes V', there are $\frac{V'}{100} DA'$ units of water and $\frac{100-V'}{100} DA'$ units of material (in which, A' is the reduced area due to consolidation); but, $\frac{100-V}{100} DA = \frac{100-V'}{100} DA'$, or,

$$A' = A \, \frac{100 - V}{100 - V'} \qquad \text{(26)}$$

and $B = \frac{V}{100} DA - \frac{V'}{100} DA'$, or,

$$B = \frac{D}{100} (AV - A'V') \qquad \text{(27)}$$

Substituting the value of A' from Equation (26) in Equation (27),

$$B = DA \, \frac{V - V'}{100 - V'} \qquad \text{(28)}$$

The rate of consolidation of the material under a given constant load, shortly after the application of the load, becomes almost uniform. It is reasonable to assume, therefore, that the average velocity of water drained during the consolidation from a percentage of voids of V to V' will be about the velocity at $\frac{V + V'}{2}$; then, from Equations (22) and (23) and for $A = 1$ sq. m.,

$$v = 150\, d^2 \, \frac{H}{D} \left(\frac{V + V'}{2 \times 45} \right)^6 \qquad \text{(29)}$$

Substituting Equations (28) and (29) (for $A = 1$) in Equation (25) and reducing the meter units to feet,

$$T = \frac{D^2 (V - V')}{492\, d^2 H (100 - V') \left(\frac{V + V'}{90}\right)^6} \quad \ldots\ldots\ldots\ldots\ldots (30)$$

It is evident from the time formula, Equation (30), that the lower the point under consideration in the Cobble Mountain Dam the longer it will take for final consolidation. Then the bottom of the dam only should be considered, say, Elevation 730.

Assuming that the average percentage of voids of the core material of the entire dam at its completion is 47.5 and that the specific gravity of the solid material is 2.72, the combined weight of water and material per cubic foot is 119 lb., corresponding to a specific gravity of 1.9, or 0.825-lb. pressure per sq. in. for each foot of head. The elevation at the top of the dam is 973; then at Elevation 730 the load will be $0.825 \times 243 =$ about 200 lb. per sq. in. The ultimate percentage of voids of the core material subject to a 200-lb. load per sq. in., is 38.15 according to Equation (24). Hence, V equals 45% at the bottom of the dam; V', 38.15%; D, 170; d, 0.008 mm.; and H, 222 (flow-line elevation = 952). Inserting the foregoing in Equation (30), $T = 735$ days.

Considering that—(*a*) there is always vertical drainage during construction; (*b*) after construction the drainage is not horizontal only; (*c*) it will take about a year for the reservoir to fill; and (*d*) during this period, if not also afterward, the drainage will take place horizontally both up stream and down stream, thus cutting in half the length of the path of percolation—it may be permissible to reduce the period of 735 days from 25 to 50%; in other words, the time necessary for the consolidation of the Cobble Mountain core may be from about 1 year to 18 months.

It should be remembered that Equation (24) is obtained with samples of an effective size of 0.008 mm. from the middle of the core. Naturally, the average effective size of the core is more than 0.008 mm., in which case the time necessary for the ultimate consolidation would be less.

As noted in Equation (30), T, number of days for consolidation, varies inversely as the square of the effective size. The average effective size for the entire core is 0.009 mm. Therefore, with an effective size of 0.009 mm. the number of days for consolidation will be, $T = 735 \left(\frac{0.008}{0.009}\right)^2 = 580$ days. This is practically within the range of the aforementioned estimated time of 1 year to 18 months.

SEEPAGE FORMULA FOR HYDRAULIC-FILL DAMS

Now that an average approximate value has been ascertained for the coefficient of permeability of the core material, the problem is to apply this information so as to be able to calculate the seepage through a hydraulic-fill dam. As far as the writer knows, there is no such general formula giving the seepage through any hydraulic-fill dam; and the derivation of such a general formula is given in the following pages.

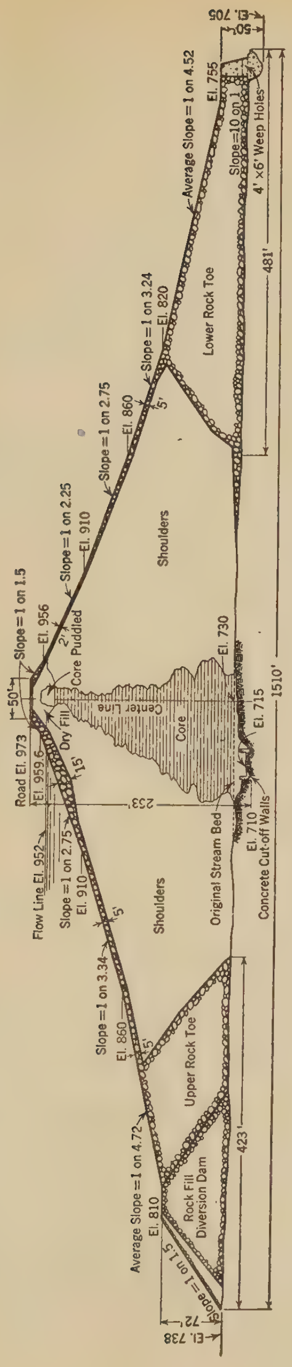
El. 705
50'
El. 755
Average Slope = 1 on 4.52
Slope = 10 on 1
4' × 6' Weep Holes
481'
Lower Rock Toe
Slope = 1 on 3.24
El. 820
5'
El. 860
Slope = 1 on 2.75
Slope = 1 on 2.25
El. 910
Shoulders
Slope = 1 on 1.5
El. 956
2'
Core Puddled
El. 730
50'
Center Line
Dry Fill
Core
El. 715
1510'
Road El. 973
El. 959.6
15'
253'
Flow Line El. 952
Slope = 1 on 2.75
El. 910
5'
Shoulders
Original Stream Bed
El. 710
Concrete Cut-off Walls
Slope = 1 on 3.34
El. 860
5'
Upper Rock Toe
423'
Average Slope = 1 on 4.72
El. 810
Rock Fill Diversion Dam
Slope = 1 on 1.5
72'
El. 738

FIG. 17.—MAXIMUM CROSS-SECTION OF COBBLE MOUNTAIN DAM.

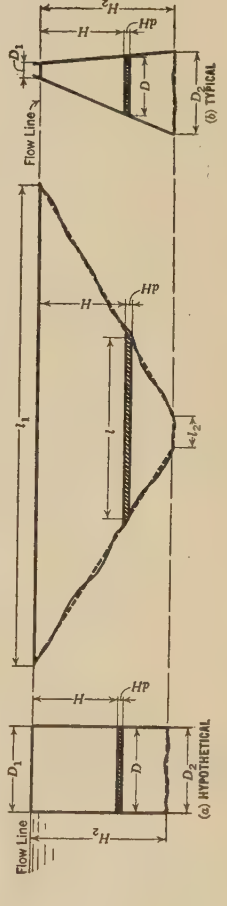
H_2
H
Hp
D_1
D
D_2
Flow Line
(b) TYPICAL
l_1
l
l_2
Hp
H
H
Hp
D_1
D
D_2
Flow Line
H_2
(a) HYPOTHETICAL

FIG. 18.—CORE SECTION.

It is proposed to use Darcy's law (Equation (19)), in deriving such a general formula. The controlling factor in the seepage through any hydraulic-fill dam, with a known or assumed coefficient of permeability of its core material, will be the section of the core of that dam, provided the core has assumed a sufficient state of consolidation to make it stable. The simplest way that Darcy's formula can be applied is to assume a horizontal flow for the seepage, although it is a fact that such is not exactly the case, at least for the upper section of the core. The diagonal flow of the seepage through the core in the upper part of the dam may result in the reduction of the core area affecting the percolation. On the other hand, there may be some clogging of the upper surface of the core resulting in decrease of percolation. However, the general seepage formula in this paper will be accurate enough for all practical purposes in estimating approximately the loss of water by seepage through the core. Fig. 17 shows the maximum cross-section of Cobble Mountain Dam.

In Fig. 18, let D = width of core = path of percolation, l_p, at H-distance below the flow line; D_1 equal the width of core = l_p, at flow-line elevation; D_2, width of core = l_p, at bottom of the dam; l, length of dam at H-distance below the flow line; l_1, length of dam at flow-line elevation; l_2, length of dam at bottom; and, H_2, height of dam to flow-line elevation.

In the hypothetical case of a rectangular core section (see Fig. 18(a): $D = l_p = D_1 = D_2$ and $l = \frac{l_1 H_2 - (l_1 - l_2) H}{H_2}$. From Fig. 18($a$), $d A = l\, d H$, and the derivative of Equation (19) is,

$$d Q = K \frac{H}{D} d A = K \frac{H}{D} l\, d H \qquad (31)$$

Substituting values of D and l in Equation (31),

$$d Q = K \frac{H}{D_2}\left[\frac{l_1 H_2 - (l_1 - l_2) H}{H_2}\right] d H$$

or,

$$Q = K \frac{l_1}{D_2} \int H\, d H - K \frac{(l_1 - l_2)}{D_2 H_2} \int H^2\, d H + k$$

Integrating between the limits, $H = H_2$ and $H = 0$, and simplifying,

$$Q = K \frac{H_2^2}{6\, D_2} (l_1 + 2\, l_2) \qquad (32)$$

Equation (32) expresses the seepage through a hypothetical, rectangular core section (Fig. 18(a)). Substituting $l_1 = l_2 = l$, then,

$$Q = \frac{K}{2\, D_2}\, l\, H_2^2 \qquad (32a)$$

Equation (32a) expresses the seepage for a length, l. In the general case of a trapezoidal core section, for any hydraulic-fill dam (Fig. 18 (b)),

$$D = \frac{D_1 H_2 + (D_2 - D_1) H}{H_2}$$

and,

$$l = \frac{l_1 H_2 - (l_1 - l_2) H}{H_2}$$

As in the hypothetical case, $d\,Q = K \frac{H}{D} l\,d\,H$. Substituting values of D and l, as before,

$$d\,Q = \frac{K\,H\,H_2\,[l_1 H_2 - (l_1 - l_2)\,H]}{[D_1 H_2 + (D_2 - D_1)\,H]\,H_2}\,d\,H$$

or,

$$Q = K\,l_1\,H_2 \int \frac{H\,d\,H}{D_1 H_2 + (D_2 - D_1)\,H} - K\,(l_1 - l_2) \int \frac{H^2\,d\,H}{D_1 H_2 + (D_2 - D_1)\,H} + k \quad \ldots\ldots\ldots (33)$$

Integrating Equation (33) between the limits, $H = H_2$ and $H = 0$, and simplifying,

$$Q = \frac{K\,H^2_2}{(D_2 - D_1)^3} \left\{ l_1 \left[\tfrac{1}{2}\,(D^2_2 - D^2_1) - D_2\,D_1 \log_e \frac{D_2}{D_1} \right] + l_2 \left[\frac{D^2_2}{2} - 2\,D_2\,D_1 + \frac{3}{2}\,D^2_1 + D^2_1 \log_e \frac{D_2}{D_1} \right] \right\} \ldots\ldots\ldots\ldots (34)$$

Equation (34) is the general seepage formula for any dam having more or less uniform slopes for the core section and the dam site. In case of any appreciable change in the slopes of the dam site, the equation can be applied in steps for different sections or heights of the dam, and their sum will give the total seepage. When $l_1 = l_2 = l$, Equation (34) becomes:

$$Q = \frac{K\,H^2_2\,l}{(D_2 - D_1)^2} \left\{ D_2 - D_1 - D_1 \log_e \frac{D_2}{D_1} \right\} \ldots\ldots\ldots\ldots (35)$$

which is the seepage for a trapezoidal core section of a dam for a length, l. Substituting $D_1 = 0$ in Equation (34),

$$Q = \frac{K\,H^2_2}{2\,D_2}\,(l_1 + l_2) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

which is the seepage for a triangular core section of a dam with different length for top and bottom. When $D_1 = 0$ in Equation (35),

$$Q = \frac{K\,H^2_2}{D_2}\,l \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

which is the seepage for a triangular core section of a dam for a length, l. When D_1 approaches D_2, Equation (34) approaches Equation (32), as its limiting value.

The difficulty with Equation (34) is that it is too long and contains logarithmic terms which make it rather cumbersome for rapid calculations. An investigation of these several seepage formulas for different geometric core sections will show that there is a relation between the seepage quantities and the areas of different core sections. This relation can be expressed as follows; let $Q_1 =$ seepage according to Equation (34); $Q_2 =$ seepage according to Equation (32); $A_1 = \frac{H_2}{2}(D_1 + D_2) =$ area of a trapezoidal core section, Equation (34); and $A_2 = D_2 H_2 =$ area of a rectangular core section, Equation (32). Then,

$$\frac{Q_1}{Q_2} = \frac{A_1}{A_2} X \qquad (38)$$

in which, X is a factor depending on ratios of $\frac{D_1}{D_2}$ and $\frac{l_2}{l_1}$.

Substituting values of Q_1, Q_2, A_1, and A_2, and solving for X,

$$X = \frac{12\, D^2_2\, l_1 \left[\frac{1}{2}(D^2_2 - D^2_1) - D_2 D_1 \log_e \frac{D_2}{D_1}\right]}{(D_2 + D_1)(D_2 - D_1)^3 (l_1 + 2\, l_2)} + \frac{12\, D^2_2\, l_2 \left[\frac{D^2_2}{2} - 2\, D_2 D_1 + \frac{3}{2} D^2_1 + D^2_1 \log_e \frac{D_2}{D_1}\right]}{(D_2 + D_1)(D_2 - D_1)^3 (l_1 + 2\, l_2)} \qquad (39)$$

Substituting values of Q_2, A_1, and A_2, in Equation (38) and solving for Q_1,

$$Q_1 = \frac{X K H^2_2}{12\, D^2_2}(l_1 + 2\, l_2)(D_1 + D_2) \qquad (40)$$

but Q_1 is the seepage in Equation (34); therefore, the general equation for any trapezoidal core section for a hydraulic-fill dam becomes,

$$Q = X K H^2_2 (l_1 + 2\, l_2) \frac{D_1 + D_2}{12\, D^2_2} \qquad (41)$$

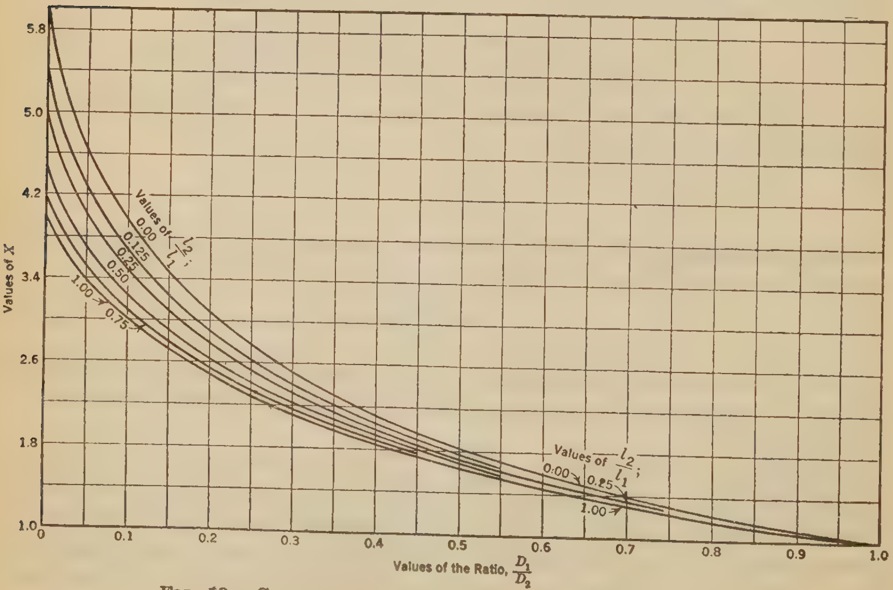

FIG. 19.—CURVES FOR SOLUTION OF EQUATION (39).

The values of X corresponding to different ratios of $\frac{D_1}{D_2}$ and $\frac{l_2}{l_1}$, can be picked from the curves of Fig. 19. It will be noted that X has a minimum value of 1 and a maximum value of 6.

Assuming that K is expressed in meters per day at 10° cent., and at 45% voids, the general seepage formula for any hydraulic-fill dam—at any percentage of voids (within working ranges), for any temperature, in gallons per day—may be expressed approximately by,

$$Q = 24.54\ X\ K\ H^2_2\ (l_1 + 2\ l_2) \left(\frac{D_1 + D_2}{12\ D^2_2}\right) \left(\frac{V}{45}\right)^5 \left(\frac{t + 10}{60}\right) \ldots\ldots (42)$$

and substituting 150 d^2 for K in Equation (42),

$$Q = 300\ X\ d^2\ H^2_2\ (l_1 + 2\ l_2) \left(\frac{D_1 + D_2}{D^2_2}\right) \left(\frac{V}{45}\right)^5 \left(\frac{t + 10}{60}\right) \ldots\ldots (43)$$

in which, Q is the quantity of flow, in gallons per day; K, the coefficient of permeability, in meters per day, at 10° cent. and at 45% voids; V, the percentage of voids varying from 30% to 60%; and the other factors are as previously defined.

ACKNOWLEDGMENT

Credit is given throughout the paper for the data used from other sources. The writer is indebted to: The late Allen Hazen, M. Am. Soc. C. E., who, before his death, read the manuscript (which then consisted of about the first half of this paper), and made useful suggestions; E. E. Lochridge, M. Am. Soc. C. E., Chief Engineer of the Board of Water Commissioners, Springfield, Mass., who, realizing the importance of such a laboratory for the construction of the Cobble Mountain Dam, allowed the necessary funds for the purpose and kept up useful interest in the work with criticisms and suggestions; the Engineering Staff at Cobble Mountain Reservoir, especially Mr. R. H. Alberti and Nathaniel Clapp, Jun. Am. Soc. C. E., who took a keen interest in the subject-matter and not only did the routine work, but also rendered valuable assistance in the preparation of the paper.

DISCUSSION

Charles H. Paul,[17] M. Am. Soc. C. E. (by letter).—The author is to be commended both for the purpose of his paper, and for the careful detail in which the proposed tests are described. Engineers interested in hydraulic-fill dam construction will find this paper valuable for study and for reference.

As Consulting Engineer during the last fifteen months of hydraulic-fill construction, the writer made regular visits to the Cobble Mountain Dam and had the opportunity to observe the ingenuity and competence of the engineers in developing practical methods of making tests for control and records of hydraulic-fill operations.

While the hydraulic-fill method of dam construction has been in use for many years, it is true that systematic study of the details of design and construction peculiar to this type has been made only within the last decade or two. Even to the present time there are few accepted standards available for investigating or testing materials, or for determining results as the work proceeds.

It is an accepted fact that there is a wide range of materials suitable for hydraulic-fill construction, as to shoulder material as well as core material. This may be seen by a study of the descriptions of the many successful dams of that type that are now (1932) in service. Progress in standardizing testing methods unquestionably will lead to more careful consideration of the requirements of design to fit the materials available. Therefore, it is quite probable that this paper may lead to an indirect result fully as important as its main purpose.

Borrow-pit sampling begins with methods generally accepted for the investigation of materials for any earth dam, but must be carried much further. Core material available is limited approximately to the quantity passing the 200-mesh sieve. An excess over apparent requirements is necessary, as some of the fines will be trapped in the beaches, and it helps core control to be able to waste some fines.[18] After the amount available has been determined and found to be sufficient, various core tests should be made for specific gravity, gradation, percentage of voids, permeability, etc., methods for which are outlined in the paper. The gradation of the coarse material in the borrow-pit should also be investigated (this may be done by screens and sieves), as the stability and free drainage of the shoulder material is fully as important as the impermeability and satisfactory rate of consolidation of the core.

The author describes the usual method of determining moisture content and percentage of voids in core and beach materials. This is a simple procedure and gives satisfactory results. His Fig. 2 shows the percentage of voids decreasing with the distance from the center line of the core. At first, the relatively high percentage of voids in the core may appear to be illogical, but that seems to be characteristic of such materials. It should be remembered that the voids in the core are microscopic, and even if they comprise a large percentage of the mass, their extremely small size renders the mass practically impervious. By reference to another section of this paper (see under "Effective Size"), it may be inferred that the rate of permeability decreases as the square root of the size of voids. Several samples of cores from the Miami Conservancy dams averaged 40% voids (weight, 125 lb. per cu. ft.; specific gravity of solids, 2.67), but the coefficient of permeability through that material was found by Glennon Gilboy, Jun. Am. Soc. C. E., to be only 0.0144 cm. per day. These void determinations on the Miami Conservancy cores were obtained from samples taken from the cores after seven years' consolidation, and, therefore, tend to confirm the results obtained by the author at Cobble Mountain. These results are further confirmed by records at many other hydraulic-fill dams.

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[17] Cons. Engr., Dayton, Ohio.

[18] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1181, 1188, and 1236.

In making and recording void determinations, care should be taken to make clear whether the amount of moisture is determined and expressed as a percentage of the entire weight (water plus solids), or of the solids alone. For example, in a sample of saturated core weighing 125 lb. per cu. ft., a water content of 25% of the mass by weight would mean 50% voids, while 25% of the dry or solid matter would mean 40% voids. There has been some confusion in the past because of lack of care in expressing or interpreting the results of moisture content determinations. The writer prefers the practice of expressing moisture content in percentage of the entire weight. That is the meaning of w in the author's Equation (1).

The writer has expressed the opinion repeatedly that rod penetration tests as ordinarily made from a boat give only indefinite results.[19] They are of some value in determining in a general way the rate of consolidation, and in detecting the presence of tongues of sand or gravel that may have worked out into the core. The author has described a more refined method of making these tests, which seems to give fairly reliable results.

Several methods of determining grain sizes and gradation of core materials have been described by Mr. Hatch, including the hydrometer method. The writer agrees with the conclusion that this method has many advantages, such as speed, low cost, convenience, and reasonable accuracy. The procedure is outlined in detail, step by step. It is hoped that this hydrometer method may be adopted for general use in the investigation of borrow-pit materials and in hydraulic core tests.

With reference to suitable sizes of core materials, it may be said in general that while the presence of impalpably fine particles makes for less permeability, it tends to increase the difficulties of construction. Character of borrow-pit materials available, study of core gradations and construction records at other hydraulic fills, and good judgment must contribute to the decision in each case as to whether the hydraulic-fill type is practical and economical.

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[19] *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1800, and Vol. LXXXV (1922), p. 1193.

In a paper describing core studies at the Miami Conservancy Dams,[20] the writer gave analyses of representative samples of the cores of those five dams, all of which averaged much finer in sizes of particles than the Cobble Mountain core. The Miami core material averaged about 0.002 mm. for effective size and about 15 for uniformity coefficient, using those terms for want of something better to indicate fineness and gradation for purposes of comparison. Observation and tests show those cores to be highly impervious, and their rate of consolidation was fast enough for all practical purposes. Samples of the core taken after consolidation averaged 40% voids and weighed 125 lb. per cu. ft. The average specific gravity of the particles was 2.67, and that of the mass, 2.00. The material of the Cobble Mountain core had an effective size of about 0.009 mm. and a uniformity coefficient of about 8. Although much coarser than that of the Miami Conservancy, this material also gave satisfactory results as regards water-tightness, stability, and rate of consolidation. At some dams where cores were somewhat finer than those of the Miami Conservancy District, more or less trouble has occurred during construction, and not many cores are much coarser than at Cobble Mountain; but between the wide limits indicated by those two, it is reasonable to assume that core materials will be satisfactory unless preliminary tests point to the contrary. The author has wisely called attention to the fact that frequent gradation analyses are necessary to obtain a representative gradation curve at any dam. The hydrometer method, as described, makes this comparatively easy and practicable. His method of plotting these curves is believed to be logical, and may well be standardized.

The author's conclusions regarding the coefficient of friction of beach and shoulder materials (see text following Equation (18)) seem logical, and may be put into practical application in the design of the shoulder sections of hydraulic-fill dams, although in many cases other considerations, such as angle of repose, may require a volume of shoulder material that will eliminate the necessity of more than a passing consideration of the coefficient of friction.

Percolation tests are difficult to make and to interpret for practical application. Nevertheless, such tests are extremely desirable in the investigation of earth dam materials and the writer knows of no more practical apparatus for such determinations than the one described as the "Large Percolator." While it may be difficult to make a direct application of the results of such tests, they certainly furnish valuable information for comparisons of different materials available, or for possible variations in gradations. These tests, either on the job or at some available laboratory, are considered to be well worth while. As would be expected, a conclusion from the author's investigation is that both finer grain sizes (which means smaller void sizes), and lower percentages of voids, have a strong influence in reducing permeability.

As to rate and manner of core consolidation, the experience with the Miami Conservancy District cores heretofore described, showed that they reached a condition very close to final consolidation in not more than twelve months after placing. The rate of construction progress has a marked effect on this, of course, since the faster the load is applied the more rapid is the consolidation process. In this connection (referring to the discussion involving Equations (25) to (30)), the writer is convinced that considerable of the core drainage takes place vertically.[21] The writer believes that this vertical drainage will have even more effect than is suggested by the author.

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[20] "Core Studies in the Hydraulic-Fill Dams of the Miami Conservancy District," by Charles H. Paul, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1181–1236.

Notwithstanding the difficulties in applying results of percolation tests to actual conditions in a dam, nevertheless such tests may often suggest limits that will be safe to apply to the dam itself. In such cases the author's seepage formula (Equation (43)) is set up in comparatively simple and convenient form for use. It should be realized also that this formula is applicable to the seepage-resisting materials in any type of earth dam, and not to hydraulic cores only.

It is the writer's belief that a well-built hydraulic core, composed of materials within the limits of fineness being discussed in this paper, will scarcely permit enough seepage to cause any concern. From the nature of their construction, hydraulic-fill dams are particularly adapted to resist seepage through the core itself, and to distribute such seepage harmlessly and effectively through the beach and shoulder materials on the down-stream side. Assuming that the core is homogeneous, what little seepage may occur will be taken care of automatically. One thing to be watched carefully is the preservation of uniformity in the core during construction, and the prevention of tongues or layers of sand or gravel that will provide a path for free seepage. Seepage formulas must necessarily assume the absence of any such conditions as that.

In many cases, also, it will be found that the dam itself is more watertight than the abutments against which it rests. Seepage dangers should be investigated through abutments and foundations of earth dams, as well as through the dams themselves, as otherwise these abutments or foundations may be the source of the greater danger.

JOEL D. JUSTIN,[22] M. AM. SOC. C. E. (by letter).—The author has performed a valuable service to the profession in making available the laboratory control methods utilized on the Cobble Mountain Hydraulic-Fill Dam. It is to be feared, however, that many engineers not previously acquainted with these methods will gain the impression, on reading Mr. Hatch's paper, that the laboratory control there practiced was unduly elaborate and expensive, and, therefore, not applicable to work which they might be contemplating.

The writer knows that such an impression would be erroneous. The cost of the laboratory equipment utilized was not great, and the total cost of laboratory control was only a small part of the cost of engineering supervision and only an insignificent percentage of the cost of the structure. Accordingly, the writer believes that it will help to promote the use of laboratory control methods similar to those utilized at Cobble Mountain, if Mr. Hatch will give some data on the cost of the laboratory, the number of men employed in this branch of the work, and the total cost of laboratory control for the job, together with the cost per cubic yard of material in the dam. Such data would also help engineers responsible for the design and construction of earth dams in persuading their principals that it is worth while and not unduly expensive to have a thorough laboratory control on such work.

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[21] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1194, 1199 (Fig. 11), and 1201.

[22] Cons. Engr., Philadelphia, Pa.

The magnitude, character of materials available, and type of dam used, should govern the degree of elaborateness adopted for laboratory control. With the advantage of the pioneer work in this line done by the late Allen Hazen, M. Am. Soc. C. E., and Mr. Hatch at Cobble Mountain Dam, and by a few others elsewhere, the necessary cost of laboratory control for most earth dams may be quite insignificant. Thus, at the Bee Tree Dam,[23] a hydraulic-fill dam, in North Carolina (177 ft. high), Charles E. Waddell, M. Am. Soc. C. E., utilized a sufficiently precise method of laboratory control for the existing conditions at a cost for laboratory equipment of less than $750, and one laboratory man and an assistant did practically all the work required.

In his Fig. 2, the author shows for a particular case, the percentage of voids in the core and in the beaches at various distances from the center line. The percentage of voids in the core is shown to exceed 50% slightly, the voids decreasing with the distance from the center line until at about 240 ft. the voids in the beaches are 25% on one side and 33% on the other side of the center line. In spite of this, the beach, because of the tremendously greater size of pores, was undoubtedly much more pervious than the core. The author does not state the depth below the surface of core pool or silt line at which the core samples were taken; nor does he state how long the core material, which had more than 50% of voids, had been in place.

One of the most compelling reasons for having a thorough laboratory control system in hydraulic-fill earth-dam construction is to obtain data that will enable one to control the stability of the core during the critical period while construction is in progress. The author gives a formula (Equation (24)) for determining the ultimate percentage of voids in the local core material at any depth, and another formula (Equation (30)) for determining the time it takes to secure this ultimate percentage of voids. He does not, however, give the percentage of voids in the core, which it is necessary to secure in order to have a stable core, nor the length of time, height, or pressure of material above, required to obtain this stable core. By a stable core is meant a condition of the core such that it is practically self-sustaining, and, therefore, does not exert material pressure against the retaining beaches or toes. The writer believes that the degree of stability of the core at different elevations during the progress of construction is one of the most important things that the engineer charged with the responsibility for the design and construction of a hydraulic-fill dam wants to know.

At the Calaveras Dam investigation, Mr. Hazen found that core material having 50% of voids exerted practically full hydrostatic pressure, but that core material having 40% of voids was sufficiently stable. The material in the Cobble Mountain core was much coarser than that at Calaveras, and it would be interesting to know at what percentage of voids the material became sufficiently stable.

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[23] "Earth Dam Projects," p. 94, John Wiley & Sons, Inc., N. Y., 1932.

Under "Seepage Formula for Hydraulic-Fill Dams," Mr. Hatch states that, so far as he knows, "there is no such general formula giving the seepage through any hydraulic-fill dam; * * *," and then proceeds to develop a formula for seepage through the core of such a dam. He is not entirely correct in this statement. A formula that may be used for this purpose was developed by Mr. Hazen, and is given by Mr. Hatch as Equation (22). The writer has previously given formulas for seepage through earth dams and also for seepage through the foundation soil.[24] These formulas are merely adaptations of the well-known Slichter formula to the conditions existing in earth dams. All these formulas, as well as that of the author, are based on the Darcy formula first presented in 1857 and given by Mr. Hatch as Equation (19).

The thoroughness with which laboratory control methods were developed and applied at Cobble Mountain have not been equalled elsewhere, as far as the writer is aware, and the detailed discussion of these methods given by Mr. Hatch will be extremely useful to all engineers engaged in the design and construction of earth dams.

JEPTHA A. WADE,[25] M. AM. SOC. C. E. (by letter).—The soil hydrometer described by Mr. Hatch is of interest where approximate results will serve and the elutriation method requiring more elaborate laboratory equipment is not justified. Apparently, judged by empirical standards, the results are consistent and close enough for ordinary purposes. By this method a good check can be kept on the materials entering a dam. It may also be used in determining whether the available dam material is suitable for hydraulic placement, or whether a rolled embankment is to be recommended due to excessively small grain size. It has been used, on work under the writer's charge, in analyzing samples from core borings in soft foundations to determine the probable seepage underneath an earth dam. Other applications will doubtless appear. Due to its comparative simplicity, both of apparatus and manipulation, it is undoubtedly a useful tool, and Mr. Hatch is to be commended for having developed the method for its use.

It is somewhat to be regretted that the paper does not go more fully into the underlying theory, with a frank statement of the factors that have been neglected and a determination or estimate of their limiting values. As it stands, the discussion is not entirely clear, considerable reliance being placed on the fact that the results agree fairly well with those obtained by elutriation.

The statement (under the heading, "The Hydrometer") of what is happening in the sample under test, is somewhat confusing—in fact, not strictly true. Assuming that the graduate contains a uniform mixture from top to bottom and that settlement takes place in accordance with Stokes' law, there will be particles of all sizes at all levels when the test is started. All these particles immediately begin to settle at rates dependent on their respective diameters. Each horizontal section will gain from above the same number of particles of a given diameter as it loses to the section beneath, the only exception being at the top where none can be supplied and the number of particles in suspension, therefore, becomes smaller. Particles originally near the bottom begin to come to rest on the bottom of the graduate. As time goes on, the available supply of particles of a given size above a given section becomes exhausted and the total grams in suspension per liter at that point is decreased. There are still particles of all the smaller sizes above this point, and of all sizes below this point.

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[24] "Earth Dam Projects," pp. 139 and 174.

[25] Chf. Engr., California Water Service Co., San Francisco, Calif.

Conditions at the various levels at any time can be determined if the analysis of the material in suspension is known. Assume that this is as given in Table 3 of the paper, or that the graduate contains:

45.58 grams smaller than 0.09 mm in diameter (200-mesh screen)
27.2 grams smaller than 0.0467 mm in diameter
19.7 grams smaller than 0.0345 mm in diameter
11.9 grams smaller than 0.0230 mm in diameter
8.9 grams smaller than 0.0165 mm in diameter
5.9 grams smaller than 0.0097 mm in diameter
4.2 grams smaller than 0.0069 mm in diameter

Suppose these quantities are plotted as abscissas marking the vertical lines between the bands with the separation sizes. Next, apply Stokes' law, determining the distance a particle of each separation size would drop in each of the times for which readings were taken. Plot these distances as ordinates measured down from the top, which represents the surface of the liquid. In 1 min each particle, 0.09 mm in diameter, will have dropped 44.58 cm. There will be no particles as large as this nearer the surface, but at greater depths the number of this size per liter will be uniform and the same as at the beginning. Particles, 0.0467 mm in diameter, will all have traveled 11.94 cm, those 0.0345 mm in diameter, 6.54 cm, etc. If these distances are plotted on the lines representing these sizes and a curve is drawn through the points, the result is a curve above which no particle of the size represented by the abscissa exists at the end of 1 min and below which conditions are the same as they were at the beginning. Since horizontal distances represent grams per liter the abscissas of the curve represent the total grams per liter at the various depths below the surface at the end of 1 min. Similar curves can be drawn for the other time intervals as shown in Fig. 20.

The hydrometer depends on the density of the displaced liquid for its operation. While there may be a question as to suspended solids increasing the density of the liquid it appears that, within certain limits at least, water with material in suspension affects the hydrometer generally as if this were true. While this suggests itself as a subject that might properly be investigated, no such attempt will be made here, the assumption being that the relation is a true one. It now appears that, in the work under discussion, the hydrometer is being used in a very peculiar manner, namely, in a liquid having different densities at different depths, increasing from the top down. What is the resultant effect? The buoyancy exerted on any floating body is equal to the weight of the displaced liquid which must also equal the weight of the floating body. The hydrometer reading indicates a summation of all the buoyant forces acting on the hydrometer; that is, each horizontal section of displaced volume multiplied by its respective density. The reading might be termed the resultant of all these forces.

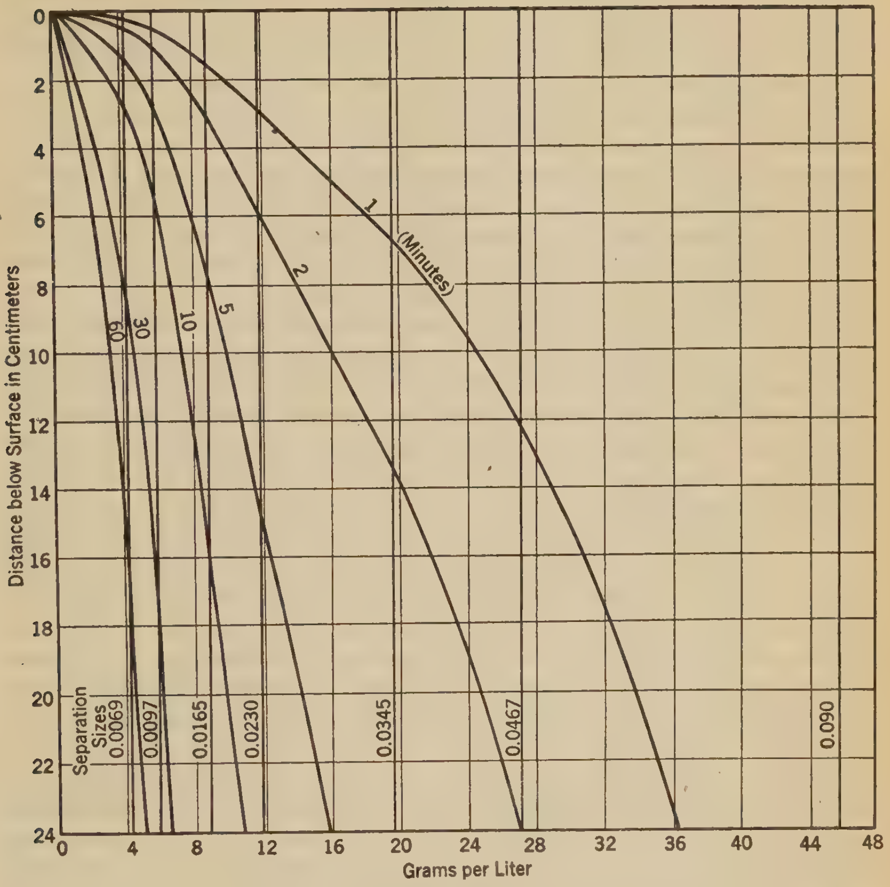

FIG. 20.—GRAMS PER LITER AT DIFFERENT DEPTHS AT TIMES READINGS ARE TAKEN. (ANALYSIS OF MATERIAL AS IN TABLE 3.)

The use of the hydrometer for analysis purposes, therefore, depends on finding the horizontal plane in which the actual grams per liter in suspension, if it was constant throughout the displaced volume, would produce the same effect as is actually obtained. Mr. Hatch assumes that this level is at the center of buoyancy of the hydrometer, but examination of Fig. 20 indicates a slight error in the assumption. Suppose, for the moment, that the hydrometer was cylindrical in form. The center of buoyancy would then be exactly half way between the surface of the liquid and the bottom of the hydrometer.

The density lines will usually be convex upward, as they are in Fig. 20, since any other hypothesis denotes an extraordinary preponderance of the larger sizes. The density at the mid-point will obviously be greater than the mean, and the density equal to the mean will be found at a higher level. Since the center of buoyancy is the center of gravity of the displaced liquid, it is probable that the concentration corresponding to the reading of the instrument will always be found at some higher point. Distances used in Column (5), Table 3, are, therefore, too great, resulting in too great velocities of fall in Column (6) and too large diameters of particles in Column (8).

It would appear impossible to evaluate this error, depending as it does on the analysis, except by "cut-and-try" methods involving a large number of computations which the results actually obtained do not seem to justify. Correcting it would increase the divergence from elutriator results pointed out by Mr. Hatch under the heading "Comparison of Elutriator and Hydrometer Method Results," and it would appear, therefore, that there are other compensating factors which have not been considered.

Another point, also of minor importance, concerns the graduations of the instrument. Dr. Bouyoucos states[25a] "the hydrometer was calibrated on average normal loam soils." Nothing is said about the specific gravity. Between the two specific gravities encountered at Cobble Mountain, there would be a difference in the instrument reading of about 1 gram per liter where the readings are approximately 60 grams per liter and proportionally smaller errors accompany smaller readings. The error here is not likely to be large, but can be evaluated easily by calibrating with material of known specific gravity. Dr. Bouyoucos also states[25a] "in peats and mucks the hydrometer method is not very reliable, first, because these materials are almost impossible to disperse and, second, because they have a smaller specific gravity." While peats and mucks are not classified as suitable dam materials, this raises another question, namely, does the specific gravity change during the process of settlement enough to effect the results? Perfect homogeneity is not to be expected.

The question of dispersing the solids properly is sometimes troublesome. One reagent may work properly with one material and may be an absolute failure with another. It would be helpful if Mr. Hatch would append a list of those that may be tried, since trial seems to be the only way of determining this vital point.

It is not felt that any of the points discussed are of enough importance to put the method in serious doubt. Rather, it is felt that discussion of them is essential to a better understanding which must precede the general adoption of the method and perhaps lead to its improvement. It is to be noted that temperature corrections have been applied in Column (4), Table 3, that are much closer than it is possible to read the hydrometer and that the viscosity is 1.00 for a temperature of 68° F. It would seem possible to control the temperature closely enough so that these corrections might be neglected, without causing important errors. This would simplify operation considerably. It is quite probable that a few check runs with an elutriator on the material from a given location will determine a fairly constant correction by which hydrometer and elutriator results may be brought into close agreement. The errors, with the exception of temperature, evidently have some relation to the analysis. Mr. Hatch very properly points out that the individual hydrometer must be calibrated, and it is believed that, if this is done and an occasional check made in a laboratory equipped for elutriation, the hydrometer may be used with entire confidence.

[25a] "A Comparison of the Hydrometer Method and the Pipette Method for Making Mechanical Analysis of Soils with New Directions," by George John Bouyoucos, *Journal*, Am. Soc. of Agronomy, August, 1930, Vol. 23, No. 4, p. 750.

STANLEY M. DORE,[26] ASSOC. M. AM. SOC. C. E. (by letter).—Engineering supervision of the construction of large dams of the earth embankment type has included first, the inspection and choice of construction materials, and, second, the regulation of the manner of incorporating the materials chosen into the structures. In recent years the soil testing laboratory has taken an important and active part in the study of materials in connection with both these phases of the supervision, and the decisions made before and during construction have been influenced considerably, and justly so, by its findings. Construction of dams, by the hydraulic or the semi-hydraulic methods especially, has been dependent upon and governed by the periodic analyses and tests of the laboratory, in order that the quality of the material in the finished structure may be ascertained. Thus, the findings of the laboratory for any such project constitute data of vital significance and utmost value to those interested in earth embankment design and construction.

The manner in which testing has been conducted on recent large projects has varied considerably, however, the engineers on each devising and inventing their own methods and apparatus to a large extent and interpretating their own data. The result has been that the findings obtained for one project have not been comparable with those for another, and for that reason many significant data have failed to be as valuable as they should to the Engineering Profession. The need for standardization of methods and apparatus is decidedly apparent. The author's appeal for such standardization is to be considered seriously and energetic action, through engineering organizations, toward that end is strongly recommended. Much has been done under the direction of such governmental departments as the U. S. Bureau of Public Roads, the U. S. Bureau of Agricultural Research, and the U. S. Geological Survey, and papers and reports dealing with the subject have been written by Messrs. Hazen, Slichter, King, Stearns, Meinzer, Terzaghi, and others. Many of these works may be of use in the standardization of soil testing, but considerable thought and discussion of the problem are urgently needed at present.

Soils, the most abundant natural construction material, have quite variable structural characteristics—even similar soils in one locality. Adequate standard terms for discussing these variable characteristics are lacking to some extent and definition of them in simple practicable terms might well constitute the first step toward standardization. Even though two materials to be compared have radically different characteristics, the existence of common definitions would permit the description of these differences so clearly that studies of the results of analyses made of them would be productive of useful data. In contrast, the lack of common adequate phraseology and definitions makes comparisons of test results futile as the true differences of the materials compared are not fully understood, simply because at present this deficiency hampers minute and clear descriptions of those differences.

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The mechanical analysis of a soil sample is probably the most widely used single test of soil characteristics, and attention is directed to the author's recommendation for the careful rating of sieves and for the accurate determination of the finer particles. For the latter he indicates that successful results can be obtained with either the elutriation or hydrometer method, and that the use of both may be advisable. In addition, it seems that considerable might be done in recording other features in connection with this analysis, such as the determination of the shapes of the grains, the ratio of the percentages of each different type of shape to the whole, the surface characteristics of the grains (whether sharp, smooth, angular, ragged, rough, etc.), and the chemical, physical, and geological make-up of all the several grains, and, where possible, their specific gravities.

Microscopic examination and measurement should be used more extensively. Too little attention is now given to these features, which greatly affect the structural qualities of the soil. When the mechanical analysis curve of a given sample is recorded graphically, as is commonly the case, the results of microscopic and other examinations and measurements might well be included on the same sheet. Then, for example, when the permeable qualities of this sample are compared with those of another similarly tested, more competent and reasonable conclusions could be reached and relations between the permeable qualities and the mechanical analyses and grain shapes might be determined.

At present, a comparison of mechanical analyses curves really gives little information of the relative structural characteristics of two samples, but only the differences and likenesses in respect to distribution of sizes of the grains. The sizes determined by sieving are not even strictly comparable as the sieves do not always separate grains from one sample of the same volumetric size as those from another, because of the effect of the shape and. surface properties of the grains upon the sizes separated during sieving. This condition is true also to some extent in the measurement of finer grains by elutriation or hydrometer methods. Briefly, it is believed that the shape of the grain has fully as much influence on the structural properties of a soil as the size, and to the present time laboratory methods in general offer no adequate means of analysis and study of the effects of the shapes of the soil grains.

"Soil engineering" has advanced with rapid strides in the past decade and probably will progress at an accelerated rate in the next ten years, so it seems that the time is ripe for consideration of standardization of testing methods.

The apparatus used in the Cobble Mountain Testing Laboratory was given considerable thought as to its make-up, and careful attention as to its performance during use. The methods of determining grain-size analyses by sieving and by hydrometer or by elutriation have similarly been used elsewhere, and as these methods and the results obtained are well described and analyzed in this paper, they might well be copied in detail for use on other projects. The tests for turbidity, moisture content, and organic content are, at present, in common use elsewhere. The apparatus described for the measurement of the coefficient of friction, although not generally used, seems worthy of consideration for adaptation for future testing as it seems to have given the required results. More definite knowledge, however, is desirable regarding the effect of moisture content on tests made with this machine as well as the effect of crushing of grains during testing.

The tests for percolation, or permeability, and for porosity, however, seem to be the most difficult for engineers to agree as to which method of testing gives the most satisfactory results. Although the author did not state which of the three methods for testing permeability (the large percolator, the standpipe permeameter, and the standard compression apparatus) used at Cobble Mountain he considered the best, he did infer that any of these three would give reliable results and that results from the horizontal and vertical capillary tube methods were unsuitable.

Without attempting to discuss the pros and cons of any of these methods, the results obtained from any two of them are not comparable, as each method introduces individual features of testing. For instance, the capillary tubes are usually packed dry; many of the can type of tests are packed tacky or moist; and, for some soils, comparisons have shown that the former will give much larger percolations. Apparently, when the samples are dried there is a breaking apart of particles or grains held closely together, and the permeability is increased thereby. Tamping the samples has a marked effect on the permeable result obtained, and the moisture content during tamping increases this effect.

To obtain field and laboratory conditions of testing that will give results of the material in its natural state and in its final position in the structure, considerable judgment must be exercised in making representative tests. Care is also needed to describe the test conditions clearly in explaining the results obtained. Mixing the sample seems to decrease its permeability. The mixing seems to distribute the fines throughout the mass more uniformly in position so that the permeability of the sample is less than in its natural state. Handling the materials to be tested influences the results in such a manner that it is practically useless to discuss various apparatus until a common method of taking and handling the samples can be adopted.

However, the type of apparatus to be used must be given consideration. Large and small cans, permeameters varying in size, large tank percolators, and vertical and horizontal capillary tubes have been used with varying degrees of success. The results obtained are not comparable because each individual type has inherent characteristics that affect the results in a significant manner. It is of prime importance that an acceptable standard type of apparatus, even though not considered perfect, be generally adopted and used to aid in comparative studies of soil permeabilities.

The term, "effective size," as used by the author, is that of Hazen, and a relation between the 10% size of the core material and permeability was determined. In perusing his results, care should be taken not to confuse this term with that of Slichter because they are not alike by definition. Furthermore, the constant factor of 150 should not be used on other work because, when found for any other materials, this factor may vary as much as several hundred per cent. from that figure. In fact, it is believed that during the testing, the author must have found wide variations from the mean used, since it is extremely difficult to establish any relation between the 10% size and permeability that will hold for a range of samples, especially with varying uniformity coefficients. The use of any formula for permeability containing the term, "effective size" (meaning the 10% size), is limited, and the results obtained by its use should be regarded only as rough estimates.

Some surprisingly simple formulas for consolidation of the core and seepage are developed in the paper. Assumptions are made, however, which affect their sureness and accuracy, although they should prove useful as guides to other estimators. Drainage through the core is probably mostly in a diagonal line, while the formulas developed are based on the assumption of horizontal flow of water. The introduction of the arbitrary "effective size" gives a term in the formulas that is likely to upset their accuracy when used elsewhere, unless considerable testing is done to determine the proper coefficients.

In general, this paper, in describing testing methods, apparatus, formulas used, and results derived, offers a particularly valuable contribution to the literature available on soil testing, and this branch of soil engineering offers a large and attractive field for engineering research and study now and in the near future.

D. P. KRYNINE,[27] M. AM. SOC. C. E. (by letter).—The publication of the test procedure during the construction of the hydraulic-fill dam at Cobble Mountain has rendered valuable services not only to engineers interested in that particular kind of work, but to all who are engaged in soil mechanics research.

The writer was much interested in the comparison of elutriator and hydrometer methods described in the paper. The curves in Fig. 6 are quite close to each other; moreover, there is no possibility of giving definite preference to either of them. Since this is so and since the hydrometer is not expensive, requiring only a short time for the performance of the test, a question arises as to whether the elutriator should be recommended for future tests of the same character.

Relative to the slight difference between the curves in Fig. 6, it should be noted that the gradation curves for the hydrometer in the lower limits, show a higher percentage of finer particles than the elutriator. According to the author's statement, logically, it should be the reverse, as some of the fine particles may go down with the larger ones.

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In the writer's opinion, since the deviation of the curves in Fig. 6 is systematic, there should be a rational cause therefor. Probably the explanation is to be found in the conventionality of the position of the center of buoyancy. Although the curves in Fig. 4 are determined experimentally, they refer to a uniform solution or suspension, while what is dealt with in an actual case, is a non-uniform soil suspension. The center of buoyancy may be determined as the center of gravity of the displaced liquid. During the experiment, coarser particles tend to occupy lower positions than the fine ones, and, in a general case, the density of the suspension increases toward the bottom. Hence, the center of buoyancy of a non-uniform soil suspension should be lower than that of a uniform one. Owing to this fact, the actual abscissas in Fig. 4 are probably under-estimated. Therefore, this should also be true of the diameter of particles that is to be computed, using Equation (8). In other words, in a general case, a certain over-estimation of the amount of the fines seems to be inevitable.

From a practical point of view the variation shown in Fig. 6 is quite negligible (perhaps 1 or 2% by weight). An ideal "size distribution curve" cannot be obtained, and it is not worth while to spend time in refined determinations. In this connection, the writer does not see very clearly why the curves of Fig. 4 should be traced so accurately. Those curves take into account one factor only, namely, the peculiarities of a given hydrometer; but they do not pay attention to the other factor, which is the non-uniformity of the soil suspension in the cylinder. In addition, the degree of non-uniformity is changing during the entire experiment. Perhaps simple straight lines could be substituted for those curves with the same degree of accuracy as far as the final results are concerned.

The author assumes that the distance of fall of a particle in the experimental cylinder is equal to the distance from the surface of the liquid to the center of buoyancy. The writer does not possess concrete data with which to contradict this statement, but it should be noted that some investigators assume a smaller value. The U. S. Bureau of Public Roads, for instance, takes the assumed distance of fall as 0.42 times the distance from the surface of the suspension to the elevation of the bottom of the hydrometer.[28]

Different fractions of a soil may possess different specific gravities; this is especially true of fine particles. Consequently, the center of buoyancy may be displaced. This fact has never been studied.

The author compares the hydrometer and the elutriator. It would be better, perhaps, to compare the hydrometer and the pipette method; actually, the latter may be considered as a more or less reliable standard. Another test which, apparently, should not be used hereafter, is the turbidity test, because, as the author states, it has lost its value since the introduction of the hydrometer method.

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[28] Reports on Subgrade Soil Studies, Reprinted from *Public Roads,* Vol. 12, Nos. 4, 5, 7, and 8, p. 69.

In discussing the penetration test the author states that "by applying the total load at the beginning of the test, the pipe would penetrate farther into the core than by applying the same total load in fractions at different intervals." The difference involved was a penetration of 2 or 3 ft less. This is a notable fact which is analogous, however, to the increase in resistance of a pile after rest, especially in impermeable soils. This fact is well known to both practical pile-drivers and soil-mechanics theorists. The author's Equation (2) is merely local and cannot be generalized.

Important and highly interesting experiments were made by the author in determining the coefficient of friction. The curves in Fig. 10 show clearly that the coefficient of friction of a soil is not constant, but that it depends on the outside pressure. Since the experimental data are not plotted in Fig. 10, it is difficult to say whether or not these curves tend asymptotically toward infinity when the outside pressure vanishes. This tendency is shown in Fig. 10 and is also required by Equation (18). Another solution would be an intersection of the axis of ordinates at determined finite heights, a result reached[29] by Charles Terzaghi, M. Am. Soc. C. E. The right side of Fig. 10 furnishes very consistent results which coincide with those of other investigators. For instance, Terzaghi[29] gives the value 0.52 for compacted Ottawa sand, the outside pressure being 10 kg per sq cm. Practically the same value is given by the author (load, 500 lb per sq in.).

Equation (1) gives the percentage of voids, based on the percentage of water by weight and the specific gravity of core material. In other branches of soil research, however, the percentage of water is always determined by dry weight. If w_0 designates the percentage of moisture by dry weight, the equation for determining the percentage of voids, would be:

$$V = \frac{100\ w_0\, s}{w_0\, s + 100} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

Obviously, the results computed according to Equation (1) and Equation (44) should be the same.

Another difference is in plotting results of mechanical analysis. In practically all papers dealing with dams, the coarser particles are at the right side of the diagram, as in Fig. 8; while the "size-distribution curve" used in some other research branches is constructed exactly in the opposite manner. Soil mechanics is a young science, and small discrepancies similar to those described, should be reconciled immediately.

As to the seepage formulas, there are not only formulas, but books, written on that subject. To the writer's knowledge, there is a German work, that deals with experiments on the moisture movement in the interior of an earth dam and in which attempts are made to express the results analytically.[30] Another book printed in Russian[31] is an interesting mathematical treatise discussing seepage phenomena in earth dams.

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[29] "Handbuch der physikalischen und technischen Mechanik," Edited by F. Auerbach and W. Hort, Vol. IV, Pt. 2, p. 523, Leipzig, 1931.

[30] "Die Wasserbewegung in Dammkörper," by Ignaz Schmied, 1928.

[31] "Seepage Through Earth Dams on Impervious Foundations," by Prof. N. Pavlovsky, 1932.

M. M. O'SHAUGHNESSY,[32] M. AM. SOC. C. E. (by letter).—The author is to be commended for the various devices developed in procuring information as to the texture of the core materials for this phenomenally high composite dam. A very rational method was adopted in constructing substantial toe-walls of loose rock, which are a valuable asset to any kind of an earth dam. The core material is of particularly thin section for this type of dam. The concrete toe-wall is a substantial structure, but the writer questions the wisdom of leaving such large weep-holes as 4 by 6 ft in size. A greater number of smaller weep-holes, with clean porous gravel, are far more effective.

The dam material was discharged from the mud line, the larger particles settling first, and then smaller and smaller particles until the very finest formed the core pool. The very fine particles held in suspension ultimately make up the core or water-tight section of the dam.

The Cobble Mountain Dam is the first of this type built in the East. The practice of hydraulic-fill construction has been developed successfully in the West for a great many years. This dam is a unique structure, pioneer of its particular type, and is entitled to all the success its projectors anticipate.

JOEL B. COX,[33] ASSOC. M. AM. SOC. C. E. (by letter).—The author has done a most praiseworthy piece of work in presenting the data derived from the testing program of the Cobble Mountain Dam to the profession. The time is ripe for a thorough-going consideration of the principles of hydraulic-fill dam construction, and of the possibilities of scientific design based on known physical facts. The establishment of adequate laboratory procedure is essential to this undertaking. The writer presents the following discussion in order that engineers may also have available for comparison the results obtained from a testing program at an entirely different dam, dealing with a material almost diametrically opposite in its physical characteristics from those met with at Cobble Mountain. In its scope and intent this program was considerably more extensive than that at Cobble Mountain, or than has heretofore been attempted in connection with hydraulic-fill construction. Following the precedent of the author's paper, a full report on methods used and results obtained, by Maitland C. Dease, Testing Engineer, McBryde Sugar Company, Limited, has been filed for reference with the complete manuscript of the author's paper in Engineering Societies Library.

The testing program at the Alexander Dam, Wahiawa Valley, Island of Kauai, Hawaiian Islands, completed in 1932, was developed in the belief that recent advances in the science of soil mechanics, and most notably, the work of Charles Terzaghi, M. Am. Soc. C. E., were sufficient to allow a complete analysis of the stresses within a hydraulic-fill structure and to enable design and control of construction to be based on such an analysis.

Professor Terzaghi's work[34] forms the starting point of the theoretical developments obtained in the technical control of the Alexander Dam. Early in the investigation it was found that tests did not always follow closely the rules laid down by Professor Terzaghi. The first addition to Professor Terzaghi's theory necessary to account for the observed facts was to assume that there were inter-particle forces of attraction even if the material was saturated and immersed in water.

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[32] Cons. Engr., San Francisco Public Utilities Comm., San Francisco, Calif.

[33] Civ. Engr., McBryde Sugar Co., Ltd., Eleele, Kauai, Hawaii.

[34] "Erdbaumechanik auf bodenphysikalisher Grundlage," by Charles Terzaghi, M. Am. Soc. C. E., Leipzig und Wien, 1925.

Shearing resistance was, therefore, a compound phenomenon, being partly true cohesion, due to these little understood particle attractions and partly apparent cohesion and friction, a frictional effect caused by external load. This external load, in turn, could consist of a mechanically applied load or of a load applied by the surface tension of the water films at an external air boundary.

This recognition of true cohesion has also been made under Professor Terzaghi both by the U. S. Bureau of Public Roads[35] and at the Massachusetts Institute of Technology.

Later, it has been shown that the grains of soil cannot be considered as perfectly elastic bodies in the compression phenomena of a clay-like soil, but that their deformation is governed partly by complex frictional effects and partly by laws of plastic flow.

Other phenomena of great importance in the actual behavior of soils under stress arise from the inclusion of air in various states in the mass and in the effects of base replacement and colloidal effects due to absorption and the change in physical character coincident with a change in the acidity of the soil.

In many such relationships, it is apparent that the mineralogy of the soil, the actual chemical constitution of the various individual grains, is of the utmost importance. As yet no completely satisfactory methods have been devised for the identification of, or for experimentation with, such minerals, and much progress in the understanding of soil phenomena may be expected along these lines. No chemical analysis of a soil as a mass can be of final value without knowing how the elements are distributed in the various grains of different sizes.

In all this work, it has been apparent that great advantage to all concerned will result from a closer knitting together of the work of those who are studying soils from widely different angles. The agricultural soil physicist, the engineer concerned with soil, and the soil mineralogist and chemist can each learn much regarding his own problems by a closer contact with the work of all the others.

Specific Gravity Determination.—The pycnometer method adopted at the Alexander Dam consisted of drying the specimen 24 hours at 105° C in a pycnometer flask, adding distilled water and boiling it for 8 hours to eliminate dissolved, entrapped, and absorbed air. By using an excellent chemical balance, carefully tared pycnometers, accurate thermometers, and care in handling, results accurate to four figures can be obtained. Simple displacement methods were frequently in error as much as 15 per cent. As all subsequently determined constants are correlated with voids-ratio, an accurate knowledge of specific gravity is most important, and engineers should be warned of the difficulties and dangers of approximate methods.

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[35] *Public Roads*, Vol. XII, pp. 89 and 117.

Permeability.—Capillary flow methods of determining permeability proved unsatisfactory at the Alexander Dam as well as at Cobble Mountain. As is apparent from the author's Fig. 15 and all other work with the consolidation of clay-like materials, the simultaneous determination of voids-ratio and permeability is necessary. No permeability determination for which the corresponding voids-ratio of the material is unknown, is of value. The criticism of the large percolator shown in the author's Fig. 14, is that the exact determination of voids-ratio is difficult, if not impossible, and, in fact, it will not be uniform throughout the depth of sample. The criticism of the standpipe permeameter (the author's Fig. 13) is that it is difficult to load with stiff and well compacted material, at the same time keeping the material homogeneous throughout. A single loading gives a determination at one voids-ratio only.

Both these difficulties are eliminated by determining the permeability in the machine devised by Professor Terzaghi on a sample for which the voids-ratio is known accurately at all times.

The clay-like laterites involved in this work may be described as soft, usually reddish, residual decomposition products of basaltic lavas or pyroclastics. They have a low percentage of silica (about 22), and are correspondingly high in iron and alumina. The specific gravity of the soil particles averages 3.2, but for individual particles the value varies widely.

Such soils are characterized by exceptionally high voids-ratios. The volume of water in such a soil is usually twice that of the solid phase even when compacted under heavy pressure. No complete explanation of this phenomenon, and of the wide range and erratic variation of this feature within a group of apparently similar soils, is available. A part of such variation is caused by the past history of the soil relating to its moisture and pressure; and compacting under pressure is shown to be a slow process which will continue to a surprising extent long after existing theories would predict a state of equilibrium. The relationship between permeability and voids-ratio is not even approximately that given by Professor Terzaghi's theory,[86] but is such that it can often be represented by the formula:

$$\log_e K_0 = c_5 + c_6 E \ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

in which, K_0 is the reduced coefficient of permeability $= \frac{K}{1 + E}$; K, the coefficient of permeability; E, the voids-ratio, or ratio of voids to volume of solid material; and, c_5 and c_6 are constants.

When plotting log K_0 against E on semi-logarithmic paper, tests on many soils will plot as straight lines, while others are concave upward and still others, concave downward. The relationship between pressure and voids-ratio follows very closely Professor Terzaghi's equations, thus,

$$E = -a_1 \log_e (p + p_c) - a_2 (p + p_c) + c \ldots\ldots\ldots\ldots (46)$$

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[86] "Erdbaumechanik," pp. 120 *et seq.*

in which, a_1, a_2, p_c, and c are constants, and p is the unit pressure on clay, or, in approximate form,

$$E = - a_1 \log_e p + c_1 \qquad (47)$$

Usually, Equation (47) is a workable and closely approximate expression for soils.

It is interesting to compare Equations (46) and (47) with the relationships found by the author at Cobble Mountain. Equations (21) and (24) may be written, using the present nomenclature:

$$\log_e (1 + E) K_0 = c_7 + c_8 \log_e \frac{E}{1 + E} \qquad (48)$$

and,

$$\log_e \frac{1}{1 + E} = c_9 + 0.0428 \log_e p \qquad (49)$$

It would be interesting to try both sets of formulas on the original data to see which actually produces the better fit. There is little actual difference over a moderate range in voids-ratio.

It is in the investigation of chemical content, or mineralogy of soil grains, that methods and knowledge are altogether inadequate. The Hawaiian soil consists of an enormous variety of separate minerals. The specific gravity of the separate grains of a representative sample varied from 5.2 to 2.2. The identifiable grains range from magnetite to montmorillonite. It is clearly impossible to obtain much of value from the usual over-all or average determination of the chemical composition of such a soil. The most hopeful avenues of attack are to determine optically all those translucent particles for which such methods are applicable and to sort out the remainder by the use of the Thoulet solution (a mixture of solution of mercuric iodide and potassium iodide that can be given any specific gravity not exceeding 3.2) into convenient separates having approximately the same specific gravity. It is probable that further study of these separates will require the use of microchemical analytic methods.

Much of interest and importance both theoretically and practically was learned by careful microscopic examination of the soil. Many of the physical qualities most important from the standpoint of the engineer, such as crushing strength, flocculation or dispersion, and permeability can be estimated by eye by comparison with similar solids the characteristics of which are known.

A method of mechanical analysis or grading by size was developed using the direct measurement of particle diameters under the microscope. A comparison of the results of such determinations using sedimentation methods is most interesting. In every case (as is to be expected) the direct measurement gives larger grains than is computed from the sedimentation data, but the extent of the difference is very revealing of the character of the soil (for example, see Fig. 21).

The differences between the two methods of determining particle size arise from the following causes:

(1).—Visible Diameter Against the Diameter of the Equivalent Sphere.—The microscopic method measures not more than two diameters, which (due to the tendency of flat grains to orient themselves) will usually be the two greatest diameters of the grains. The sedimentation method determines the diameter of an equivalent sphere which will settle at the same rate through still water. Any tendency to irregularity or flatness in the grains will be reflected in the difference between micro-analysis and sedimentation.

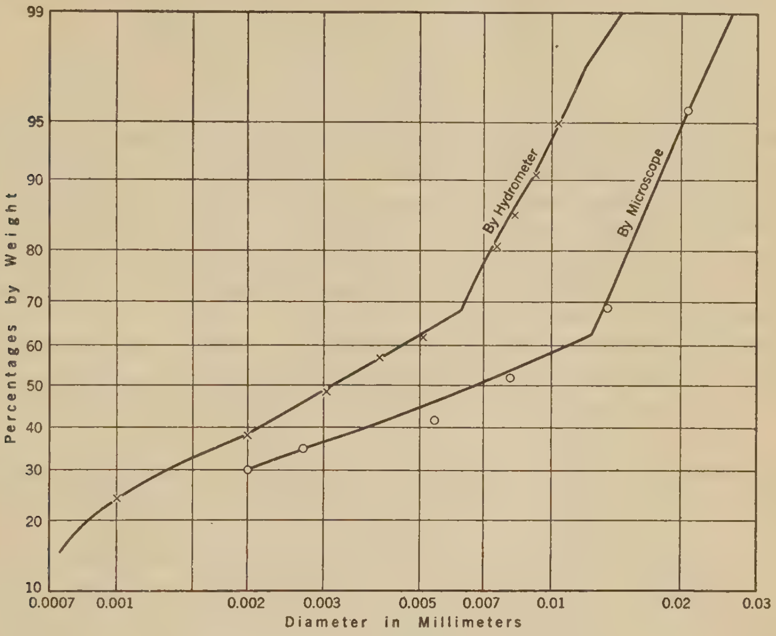

FIG. 21.

(2).—Erroneous Assumption of Specific Gravity of Particles.—The size of the particles as computed from the sedimentation analysis is governed by the specific gravity assumed. As previously pointed out, this specific gravity may differ greatly in various grains, and, therefore, any individual grain (the specific gravity of which is unknown) may differ greatly in size from that computed by sedimentation analysis, using the average specific gravity of the material.

(3).—Expanded Particle.—The excessive voids-ratio of such soils as have been studied may be due in part to excessively hydrated or expanded particles, the water of expansion of which is driven off at a low temperature and, hence, does not appear in the specific gravity determination at 110° C. Such particles will measure much larger than their speed of descent indicates.

(4).—Failure of Experimental Conditions to Fulfill Assumptions of Stokes' Law.—The sizes in sedimentation analysis are computed on the assumption that Stokes' law for the velocity of descent through still water is strictly

fulfilled. It has been demonstrated[37] that this is not strictly true for soil particles. Mechanical and electrical interactions between the particles themselves and the sides of the container give non-vertical paths, while there is a certain amount of interference from the hydrometer bulb. Slight differences of temperature set up convection currents in the liquid, and flat particles will tend to plane from side to side rather than fall vertically.

All these effects act in the same direction, to slow down the descent of the particle and indicate a smaller size than the correct value.

The really excellent check between the sizes of the sharp, hard, Kilauea volcanic sand furnishes a measurement as to the effect of Cause (4) and shows it to be small. The difficulty listed under Cause (2) will not be apparent in the average of a large sample unless a certain mineral is both lighter and tends to form larger particles than the average. The greater part of the discrepancy between the two methods of measurement appears to be conditions included in Causes (1) and (3) and both causes are significant in the physical characteristics of the material.

It was found that the relationship between permeability and voids-ratio of each one of these soils followed to a considerable degree of approximation, an expression of the form:

$$\log_e K_0 = c_5 + c_6\, E \ldots\ldots\ldots\ldots\ldots\ldots\ldots (50)$$

in which, K_0 is the reduced coefficient of permeability, in cubic meters per minute $= \frac{K}{1 + E}$; E is the voids-ratio; and c_5 and c_6 are constants.

Semi-logarithmic plotting of values of K_0 and E are shown in Fig. 22. It will be noted that, in most cases, the points fall on a straight line. A few samples give curves slightly convex downward.

Professor Terzaghi's early work on compression and consolidation of clays developed the fact that these phenomena were largely controlled by the hydraulic flow through the interstices of the material, and, at first, the time-lag of the compression resulting from the application of a given load was thought to be entirely due to this cause. Later (1928), the syllabus of the Massachusetts Institute of Technology[38] recognized that "the time effect due to low permeability combines with the time effect due to internal friction." By noting that the permeability effect varies with the square of the reduced thickness of the layer of soil, while the time effect due to internal friction is assumed to be independent of this thickness, a practical method of disentangling the two is developed. The research work at Alexander Dam, through two long-continued tests, gives additional information on this problem and suggests that the phenomenon is more complex than at first appears. One test compared the compression resulting from repeated applications of the maximum load and the rebound curves after the removal of this load. The resulting hysteresis loops are quite similar, but show a steady reduction

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[37] Experiments by F. J. Viehmeyer, Prof. of Agri. Soil Physics, Univ. of California, Agri. School, Davis, Calif.

[38] Course in Soil Mechanics, Pt. I, "Soil Physics," by Charles Terzaghi, M. Am. Soc. C. E., 1928.

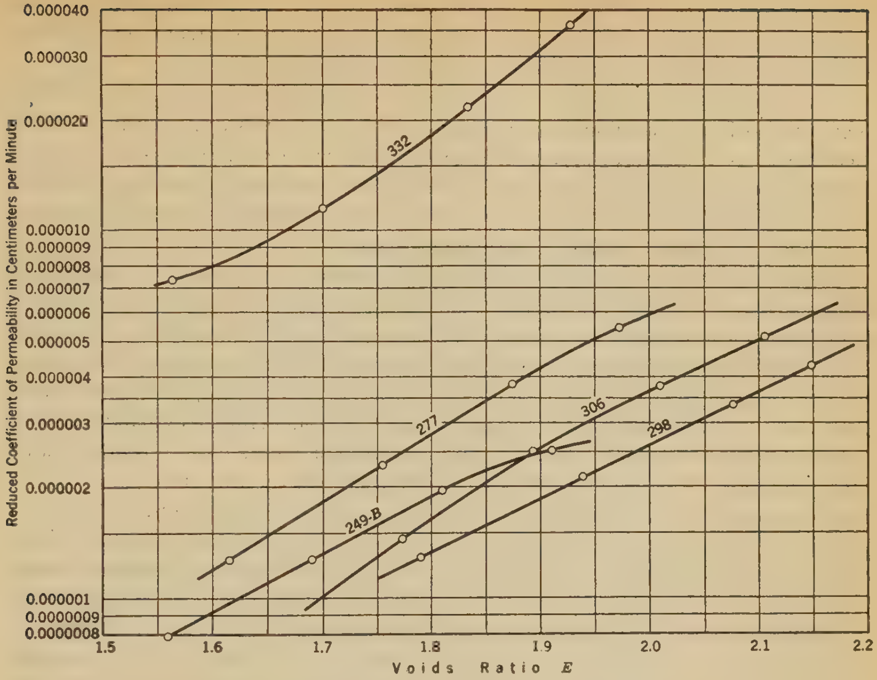

FIG. 22.—RELATIONSHIP BETWEEN VOIDS-RATIO AND PERMEABILITY OF CORE MATERIAL, ALEXANDER DAM.

of voids-ratio with successive loadings. The second test shows the slow steady reduction of the voids-ratio over many months of steady application of the same load (see Fig. 23). The results indicate that the two effects are due

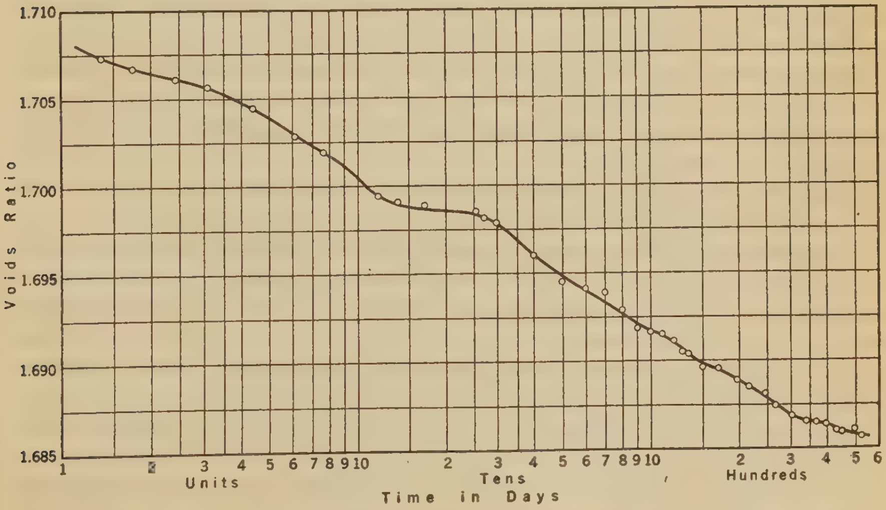

FIG. 23.—COMPRESSION UNDER THE STEADY APPLICATION OF A LOAD.

to the same cause and that the fluctuation of load is not necessary to cause increased compression. The magnitude of the long-term effect and the long period over which it continues, are most noteworthy.

The latest development in the mechanics of clay-like soils is in the related phenomena of internal friction, cohesion, and lateral pressure. Although both shearing tests and lateral pressure determination were made with simple, inexpensive apparatus (not comparable with the elaborate shearing devices developed by the Massachusetts Institute of Technology), the resulting series of values are interesting and appear significant.

For every sample for which a complete series of values is available the shearing pressure depended only on that portion of the total direct pressure which was borne directly by grain-to-grain contacts. The surplus direct pressure which was carried by hydrostatic pressure within the voids of the clay did not influence the results. In other words, if a clay sample is first tested when it is in equilibrium with its applied load, the shearing strength developed in this condition is the greatest possible without a change in voids-ratio and water content. To increase the load without allowing time and opportunity for the excess water to escape will not increase the shearing strength.

Tests of clay at moisture contents below that corresponding to equilibrium with the direct load were more erratic. At times, the shearing strength would be as great as if the full equilibrium pressure were applied. At other times, the strength would be materially less, approaching a strength directly proportional to the load. The explanation appears to be that if a clay sample contracts by drying without cracking, its interior is subjected to a pressure caused by the surface tension of the water between the grains great enough to cause it to be in elastic equilibrium; in other words, its voids-ratio is that corresponding to its actual internal pressure. Such a lump will have a shearing strength governed by this actual internal pressure, which may be many times that of the applied load. However, such a mass is too dry to be trusted not to crack, it is no longer truly plastic, and it does not always retain this unfractured state within the testing machine. Any break in the continuity of the water within its voids will release this pressure along the air and water interface, and then the shearing strength will be governed by the local grain-to grain pressure which (along this interface) is only that caused by the applied external load.

The measurements of the lateral pressure of clay with variable moisture and with varying direct pressures are likewise important and interesting. The apparatus available permitted enough water to leak past the main piston and the pressure plunger to render determinations at moisture contents higher than that at which the material was in equilibrium, with the applied load somewhat difficult and unsatisfactory. However, enough results were obtained to indicate that Professor Terzaghi's theory, that the hydrostatic pressure within the voids caused an independent lateral pressure additive to that caused by the grain-to-grain component of the applied load, was correct. This simplified the experimental problem to the determination of lateral pressure when the material was in equilibrium. To facilitate this the apparatus was rebuilt with a drainage filter in the base, and the material was loaded somewhat wetter than required and allowed to reach equilibrium in the machine. The results obtained showed a closely linear relationship between equilibrium or grain-to-grain pressure and the resulting lateral pressure. The total lateral pressure is, therefore,

$$L = Kp + w \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

in which, L is the total lateral pressure; p, the grain-to-grain pressure within the soil mass; w, the hydrostatic pressure in water in voids of the soil mass; K the constant of lateral pressure; and $P = p + w$, the total pressure in the soil mass caused by applied load. The values of K obtained with soils at Alexander Dam, and using the apparatus available, ranged from 0.35 to 0.50.

A most important relationship with clays is that which occurs between shearing strength and lateral pressure. As shown[39] by Professor Terzaghi, the lateral pressure should be given by the following equation:

$$\frac{L}{P} = \tan^2\left(45^\circ - \frac{\phi_0}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

in which, ϕ_0 is the angle of internal friction, or $\tan \phi_0$ is the value of the shearing coefficient. The only complete series of tests on materials at Alexander Dam in which satisfactory data were obtainable on strictly duplicate and comparable samples both in shear and in lateral pressure, are given in Fig. 24.

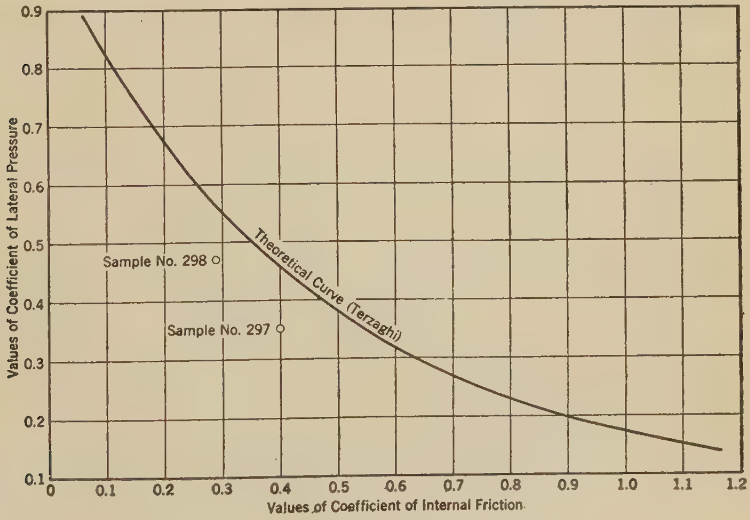

FIG. 24.

In deriving Equation (52) by means of Mohr's circle of stress, the shearing stress used must be closely scrutinized. The shearing stress developed to resist the formation of lateral pressure will be the true maximum shearing strength of the material. Not only is this the true maximum on any plane, but it is the passive or static stress. The shear measured in any test is probably not quite this pure shear, but a complex phenomenon with changing areas and distorted lines of pressure. Besides, this difference between the true maximum shearing stress which governs the formation of lateral pressure and that shown in shearing tests, there is a possibility that the internal friction of a clay, like the friction between solid surfaces, has two values, one for static, and the other for moving or dynamic, conditions. If so, the movement of the slide allowed by pure compression of the material may be sufficient to prevent a true measurement of the static coefficient of shear.

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[39] "Erdbaumechanik," p. 188.

The physical properties of soils characterized by a granular, or a crumb-like, structure are transitional between those of clays, or colloidal-cohesive soils, and clean sands. These properties are, therefore, more complex and more variable than those of either sands or clays, and, in general, the best understanding of such phenomena is obtained by a comparison between the observed action of the crumb-like soil and the corresponding performance expected from clay on the one hand and sand on the other. It is probable that soils may be found to vary by imperceptible degrees in a continuous series from the most perfect of clays to the most cohesionless sands. The contribution of any series of tests on such soils as those at the Alexander Dam is, therefore, more likely to be of practical rather than of theoretical value and to be restricted largely in its application to nearly identical materials.

Especially to be noted is the great importance in the testing procedure of certain peculiarities of these soils, such as the great effect on shearing tests of arching within the apparatus and of compression phenomena inextricably connected therewith. The great influence of such phenomena led to a design of a testing machine and a manner of conducting tests intended to allow the estimation and correction for such effects rather than their removal. To remove arching effects very appreciably would necessitate a sample of such extremely thin section as compared to its area of shearing plane that it would lead, on the one hand, to a screening of the material to remove large lumps and particles which would render the resultant material altogether unlike that actually used in the dam, or, on the other hand, to a size of testing machine quite impracticable. Therefore, the other alternative of making control tests in which the variation of direct pressures throughout the mass of soil was measured, was followed.

The importance of compression in shearing tests is not usually emphasized in accounts of such phenomena. As actually performed shearing tests are made ordinarily on comparatively small samples either in single or double shear. As shown in Fig. 25, a considerable movement of the slide takes place by compressing the material in the darkened areas, with the toggle or arching effect of these sections tending either to increase the direct pressure (if the arrangement of slides is rigid), or to force back the material above or below (if a constant direct pressure is maintained). Before a true shearing failure takes place the shearing surface is considerably reduced in area and the distribution of stresses is complicated far beyond easy analysis. The shearing surface is no longer plane. Attempts to force shear to occur along a plane surface by grids or other mechanical restraints instead of solving the problem, only complicate it further and introduce enormous uncertainties as to the distribution of the direct pressures. Much more elaborate and thoughtful experimentation is necessary before the measurement of shearing strength may be considered as satisfactorily obtainable.

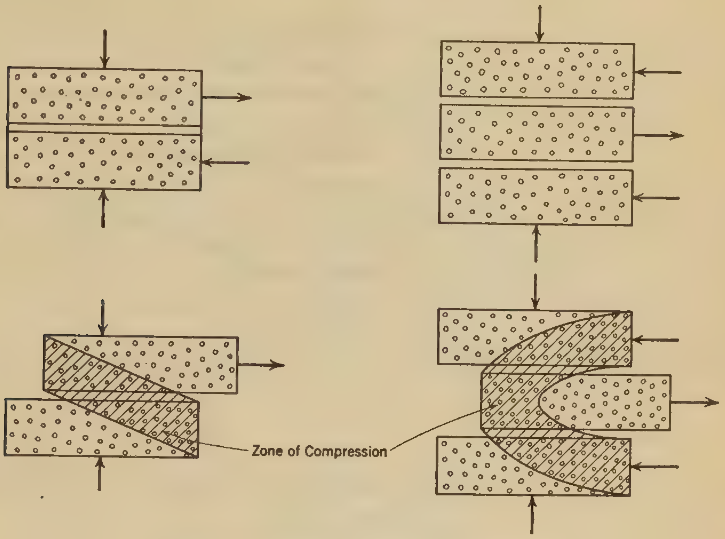

FIG. 25.—COMPRESSION DURING SHEARING.

An interesting series of tests on crumb-like soils was made to show the relationship between cohesion, moisture content, and time and pressure under which the material is packed and the cohesion developed. These are shown in Fig. 26. Cohesion was measured by means of shearing tests at light loads with an estimated correction for friction as determined from tests on the same material at varying pressures. The shearing force required, therefore, is extrapolated to zero direct pressure.

Referring to Fig. 26, the slow increase in cohesion with time after two or three days (Curve *B*), the rapid increase with pressure (Curve *C*), and the existence of moisture content both above and below which the cohesion is reduced (Curve *A*), are all clearly shown. These curves were determined by tests of the same sample under different conditions; for example, for Curve *A*, the sample was packed 2 days under a pressure of 4.97 kg per sq cm (71 lb per sq in.); for Curve *B*, the sample was kept under the same load (4.97 kg per sq cm) during the test, at a constant moisture content of 43%; and for Curve *C*, the sample was kept at a moisture content of 43% and after 6 hours was tested under various loads.

To correlate the results of compression tests in the large Terzaghi apparatus with settlements and changes in permeability observed in the dam, it is necessary to allow for the influence of larger lumps than can be tested within this apparatus. As an aid in such consideration a series of crushing tests of 2-in. cubes cut from such lumps, was run under a variety of moisture conditions. In general, an optimum moisture content is not observable with these samples. Some of them showed such an increase in strength with moderate drying, followed by a decrease as drying continued, but most samples showed a steady increase in strength down to the limit of drying obtainable in dry air. In all cases the moisture content was much lower than can be expected within the bank. Cubes cut from the soil within the borrow-pit varied in their response to soaking from those that slaked upon immersion to those showing a difference of crushing strength of 56.7% between the saturated and air-dry condition. Lumps from the Alexander Dam varied from 57% to 86% reduction in strength. Random samples showed a non-systematic variation in strength from 20 lb per sq in. to 100 lb per sq in., when soaked.

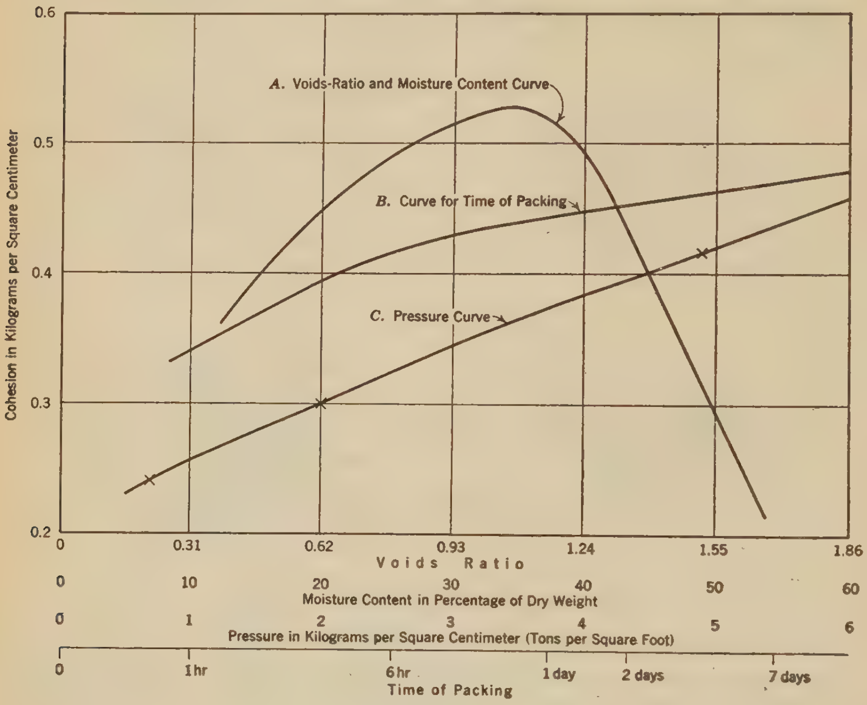

FIG. 26.

It is apparent that the settlement and compacting of such a body of earth as the beach areas of this dam, takes place not only by the compression of the finer-grained material, but also by the crushing of the coarser lumps.

The consolidation formula proposed by the author (Equation (30)) for a dam subsequent to completion is defective in its derivation. Equation (29) is based on the statement that "the rate of consolidation of the material under a given constant load, shortly after the application of the load, becomes almost uniform." This unfortunately is far from the case. This problem has been given its theoretical outline by Professor Terzaghi and has had two solutions proposed. One by Glennon Gilboy, Jun. Am. Soc. C. E.,[40] is exceedingly neat in its mathematical derivation. Its major defect is in neglecting the effect of gravity of the movement of the escaping water. The second method is one proposed by the writer for use in computing the consolidation of a dam core during construction.

It consists in separating the cross-section of the dam into a series of finite divisions and applying Darcy's law to each division separately for a short interval of time. Both divisions and time intervals must be chosen small enough to allow an average uniform velocity to be used without the introduction of too great errors.

A demonstration of the relative amount of the errors occasioned by the assumption of constant rate of consolidation is given by computing the rate of consolidation of the Cobble Mountain Dam by Professor Gilboy's method.

Taking an elliptical cylinder (semi-major axis, 5 930 cm, and semi-minor axis, 4 780 cm) that has an area roughly equivalent to that of the dam core, and a ratio of diameters about that of height to one-half the width of the actual core; and assuming an average percentage of voids of 42.8, with a Terzaghi coefficient of consolidation of 1.4, gives the following rate of consolidation:

Percentage of consolidation	Time, in days	Percentage of consolidation	Time, in days
0	0	60	501
10	21.8	70	705
20	62	80	1 010
30	134	90	1 550
40	226	95	2 085
50	365		

This is rapid consolidation, indeed, owing to the relatively coarse material of the core and the extremely narrow cross-section obtained. Even so, however, if it is estimated that 50% of the consolidation will occur during construction, the percentage of consolidation at various times after completion may be estimated, as follows:

Years after completion	Percentage of consolidation	Years after completion	Percentage of consolidation
0	50	3	88
1	71	4	93
2	82	5	96

The slowing down in rate of drainage is very marked, the drainage during the third year being only 29% of that during the first.

The writer takes exception to the argument presented under the heading "Consolidation of the Core of Cobble Mountain Dam," purporting to discredit the use of consolidation computations. Seven points are made, which will be taken up in order:

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[40] "A Theoretical and Experimental Investigation of the Properties and Behavior of Hydraulic-Fill Dams," Thesis presented to Mass. Inst. Tech., in 1928.

(1) There is no reason why loads cannot be applied in the laboratory in any way desired, and, hence, they can be arranged to duplicate conditions in the dam.

(2) The initial percentage of voids can likewise be made anything desired when conducting laboratory tests.

(3) Two questions are raised in this connection. The first is that laboratory tests thus far have been on small-sized test pieces. The largest to the writer's knowledge have been those in the large machine at the Alexander Dam, 8 in. in diameter. These samples checked admirably with similar ones tested in the smaller machines, provided consolidation controlled by hydraulic action is separated from that controlled by internal friction. The writer hopes that more work can soon be done on large-scale and long-time tests, which will give a final answer to the question. In the meantime, nothing has yet been shown sufficient to render computations based on Professor Terzaghi's hydraulic theories so seriously defective as to be valueless. The second point is the great variability to be found in small samples of apparently similar soil. This is a fact encountered throughout the realm of soil testing, and can only be met by such a number of individual tests that their average will represent fairly that of their large-scale prototype.

(4) The rough surfaces of the beaches go down with the core as it compresses and, hence, do not retard its consolidation. The effect of abutments is certainly to complicate the process in close proximity to them, but a computation near the center of the dam will usually be sufficient for engineering purposes.

(5) The time intervals for the hydraulically controlled portion of the consolidation may be exactly computed for strict similarity. Slow re-adjustment over a long time is shown by the Alexander Dam tests (Fig. 23). This argument is precisely similar to the first point in Argument (3).

(6) The drainage facilities are capable of investigation and allowance may be made for them.

(7) The variable pressures and stresses are susceptible to computation.

Table 5 presents in compact form the great contrast in physical properties presented by the core materials of Alexander Dam and Cobble Mountain, and may well explain why methods applicable to one type will not be usable with the other.

In conclusion, the writer wishes to emphasize four points, one of which is in direct accord with the author's views while three present modifications:

(*a*) Laboratory and field determinations of the physical constants of soils are already established on a sufficiently theoretical and practical basis to serve as useful tools for the designing engineer.

(*b*) Much further investigation is urgently needed in the fields of scale effect, shearing and lateral pressure phenomena, soil mineralogy, and other phases of soil physics and mechanics.

TABLE 5.—COMPARISON OF AVERAGE PHYSICAL CONSTANTS OF CORE MATERIAL

Description	Cobble Mountain Dam	Alexander Dam
Effective size, in millimeters	0.009	0.00448
Uniformity coefficient	6.0	10.27
Specific gravity	....	3.117
Voids–Ratio; at Loads of:		
50 grams per sq cm.	1.05	2.25
3 250 grams per sq cm.	0.72	1.682
Compressibility coefficient, $\frac{1}{a_1}$	5.63	5.62
Rebound coefficient, $\frac{1}{A}$	....	77.93
Permeability coefficient (in centimeters per minute, or meters per day) at a load of 3 250 grams per sq cm.	0.000178	0.0000064
Constants for Change of Permeability:		
c_s, in Equation (45)	3.93	4.07
m, in Equation (21)	6.4	7.2
Consolidation Coefficient, for $p = 3\,250$ Grams per Sq Cm:		
c (Terzaghi)	1.88	0.0451
$C = [c(1+E)^2]$, for constant volumes	5.58	0.336
Lower liquid limit (percentage)	....	69.6
Plastic limit (percentage)	....	55.5
Plasticity index (percentage)	....	14.1
Moisture equivalent	....	57.9
Hydrogen-ion concentration, pH	....	5.9

(*c*) Although the writer appreciates the motive for the author's opinion that "it is time that a definite step is taken and a standard method developed in making tests for this particular kind of construction," he does not believe that any present method of conducting tests is of sufficiently proved superiority to be worthy of such elevation to the position of standard. The writer suggests that the most helpful course at present is to compare the constants obtained from each method of testing which is being followed, and to find the extent and reason for such differences as appear. It is the physical constants of the large-scale prototypes that are desired, and standard laboratory tests are of value only as they correlate with these values.

(*d*) All recent work on soil mechanics, using different types of soil, emphasizes the unreliability of depending on the size of grain, at least in the sizes determinable by sieve and sedimentation or elutriation analyses, as a criterion for estimating the physical constants of the soil as a whole. For most purposes direct laboratory determinations of the major constants of compressibility, elasticity, permeability, internal friction, and cohesion are necessary. One reason why the mechanical analysis of grain size is unsatisfactory as a criterion is that microscopic and elutriation methods cannot be carried into the range of colloidal grain sizes, and it is the colloidal content of many soils that governs a considerable number of its physical properties. Another reason is that mechanical analysis gives no insight into the realm of packing or structure, the importance of which is especially great in foundation problems, as pointed out by Casagrande.[41]

WILLIAM P. CREAGER,[42] M. AM. SOC. C. E. (by letter).—It is unfortunate that the theory of design of earth dams is not as exact as that for most engineering projects. The greatest contributing factor in this respect is the extreme variation in the character of the materials used in their construction.

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[41] "Structure of Clay in Foundation Engineering," *Journal*, Boston Soc. of Civ. Engrs., April, 1932, pp. 168 *et seq.*

[42] Chf. Engr., The Power Corporation of New York, Buffalo, N. Y.

It is obvious, therefore, that material addition to the knowledge of the behavior of earth embankments subject to the pressure and percolation of water cannot be expected without the accumulation of an enormous amount of data regarding the characteristics of such materials in existing dams, tabulated in accordance with their reactions when subjected to accurate laboratory tests.

To be of value such tests should be strictly comparable. It is hoped that a method of testing materials for dams will be developed eventually, which will be as standard as those now available for most other building materials. The author's presentation of the testing methods used at Cobble Mountain Dam is very commendable and may well serve as a basis upon which to develop a standard method.

The writer is much interested in Equation (23), particularly in regard to the exponent of V. From this equation it is seen that the author's experiments indicate the relation between the coefficient of permeability, K, and the percentage of voids, V, to be expressed by the formula,

$$K = C\,V^{6} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

Prior to the publication of the author's paper, the only authority for the relation between these two factors was that given by Professor Slichter,[43] as,

$$K = C\,V^{3.33} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

and this relation has been used extensively in textbooks. There is probably some explanation for this difference, either in the method of testing or in the class of materials used.

Harry H. Hatch,[44] M. Am. Soc. C. E. (by letter).—The purpose of tests for hydraulic-fill dams is to determine certain characteristics of materials that bear directly on the safety or the design of the structure, as mentioned throughout the paper. The standardization of tests and the development of new testing methods naturally will lead to more careful consideration of the requirements of design as pointed out by Mr. Paul, who states that,

> "Core material available is limited approximately to the quantity passing the 200-mesh sieve. An excess over apparent requirements is necessary, as some of the fines will be trapped in the beaches, and it helps core control to be able to waste some of the fines"[18].

This cannot be true in all cases and should not be taken as a general principle.

The gradations of various core samples tested at Cobble Mountain (see Fig. 8), show the wide range of the percentage of material passing the 200-mesh sieve. The curves for the upper range, which show that almost 100% of the core consists of material that passes a 200-mesh sieve, are from dams in which fines have possibly been wasted during construction. In some of the dams represented in the lower ranges, the limit being only 50% of the core made up of material that passes a 200-mesh sieve, no fines were wasted during construction. No doubt there are other successful hydraulic-fill dams with cores that contain less than 50% of material finer than a 200-mesh sieve (ignoring any waste material), in which there are not enough fines to prevent excessive seepage.

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[43] *Water Supply Paper 140*, U. S. Geological Survey.

[44] Engr., in Chg., Cobble Mountain Reservoir, Springfield Water-Works, Blandford, Mass.

The gradation tests of eight borrow-pit samples of hydraulic-fill dam material analyzed at Cobble Mountain showed that the range of variation of the material in the borrow-pits that passes a 200-mesh sieve, was about 700% between the finest and the coarsest samples.

The clay content and the distribution of particle sizes or the gradation of the material passing the 200-mesh sieve are of utmost importance.

Excess quantities of fines in the borrow-pits, greater than the requirements of the core, are not always desirable. In fact, in some cases this condition is a source of trouble and danger. Not much fine material could be wasted in the beaches. No doubt some will be trapped here and there, but in quantity it has no importance. If considerable quantities of fine materials are deliberately entrapped in the beaches, it will create a source of weakness in the structure.

For the construction of the Cobble Mountain Dam the contractor was limited to certain prescribed borrow-pits. After the first working season (1929), when the structure had been built to a height of about 60 ft, the contractor located another borrow-pit, which was much to his advantage in regard to elevation, location, depth, quantity, etc. However, the material contained excessive fines, such that the percentage passing a 200-mesh sieve was approximately the same as that of the material in the core of the dam.

The contractor was told of the restrictions in the specifications, justified for various reasons, that no excess fines could be wasted below the dam. Extensive studies were made to provide special storage basins within or outside the reservoir basin for depositing these fines. After much time and thought spent on the proposition, the idea was abandoned because of its exorbitant cost and the impracticability of taking care of the excessive fines in the proposed storage basins.

In 1930, the contractor insisted on giving this material a trial and subsequently moved about 35 000 cu yd of it. Between the heel of the dam and the entrance of the diversion tunnel, a dike was constructed near the intake, to separate the supply pool or the settling basin from the Springfield (Mass.), Water Supply System during the construction of the dam. It was hoped that sufficient fines would precipitate in this settling basin, but in spite of all precautions, some were wasted below the dam, causing much trouble.

Immediate difficulty was encountered because the core section was built faster than the beach and not enough water was left in the core pool to float the pumps. The fines did not precipitate as readily in the settling basin as had been expected and, since the muddy water was used over again, it became almost impossible to operate the plant due to the fines being carried back and forth. To proceed with the operations meant increasing the depth of the core pool which, necessarily, would increase the width of the core.

Furthermore, the fines in this material were such that the slopes of the beaches were flat. These factors combined to produce layers of very fine material within the beach section, such that the safety of the structure was threatened, and work had to be stopped. Remedying the situation, the dam was finished with material from the borrow-pits originally specified.

Each construction job has its own problems, and these problems often vary greatly, especially as to the feasibility of wasting excessive fines. At times the quantity of water, that is necessary for wasting fines, must be considered. It is of the utmost importance to know the problem; that is, to know the nature of the material available—the conditions and the restrictions governing its use—and then to design the structure accordingly.

A comparison of the gradations of core materials in Cobble Mountain Dam with those of the Miami Conservancy dams cannot be conclusive in discussing their relative consolidation and seepage without taking into consideration the respective heights of the dams and the widths of the core sections. The maximum height of the Cobble Mountain Dam is 263 ft, compared with a height of 125 ft for the Englewood Dam, the highest in the Miami Conservancy District.

Equation (30) takes into consideration the conditions after the completion of the dam. There is no dispute concerning the vertical drainage prior to completion, but no one can state definitely how much vertical drainage occurs afterward. Whatever that may be, it can properly be assumed that the drainage of the remainder of the excess water (or the ultimate consolidation), will take place approximately in accord with Equation (30), keeping in mind the assumptions and limitations given in its derivation.

Assuming that the two dams under consideration are built with identical materials and under the same conditions, and considering only the effect of the height and width of the core and the effective size of the material, Equation (30) becomes:

$$T = \frac{c}{H} \frac{D^2}{d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

in which, c is a constant. Substituting values pertaining to Cobble Mountain Dam in Equation (55),

$$T_1 = \frac{c}{263} \frac{(170)^2}{(0.009)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

Substituting the effective size pertaining to Miami Conservancy dams in Equation (55) and adopting the Miami Conservancy design of core width at any elevation equal to the height of dam above that elevation, and with the height of the Cobble Mountain Dam,

$$T_2 = \frac{c}{263} \frac{(263)^2}{(0.002)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (57)$$

The ratio of T_2 to T_1 (Equations (57) and (56)) indicates that the Cobble Mountain Dam, with Miami Conservancy design and core material, would require 48.5 times as long for ultimate consolidation as it will under the existing conditions. The fact that the core material used in the Miami Conservancy dams showed a satisfactory rate of consolidation would not necessarily mean that the same material would show the same satisfactory consolidation if used in a dam as high as that at Cobble Mountain.

Mr. Paul offers no data pertaining to the angle of repose of materials in successful hydraulic-fill dams and their outer slopes to show that "in many cases other considerations, such as angle of repose, may require a volume of shoulder material that will eliminate the necessity of more than a passing consideration of the coefficient of friction." The outer shoulder slopes of hydraulic-fill dams are built to designed grades, usually by dikes made from sluiced beach material. During construction the dikes near the outer slopes of the Cobble Mountain Dam were built from sand and gravel excavated at the discharge end of the sluicing pipes and dumped by the drag-lines. The slope of the angle of repose of these dikes was less than 1 on $1\frac{1}{2}$, and yet the average down-stream slope of the dam was 1 on $3\frac{1}{3}$ and the average up-stream slope was 1 on 3. As emphasized in the paper, the coefficient of friction is one of the most important factors in the design of hydraulic-fill dams.

Mr. Paul's remarks in regard to prevention of tongues or layers of sand or gravel in the core and the seepage through abutments and foundations are very important. However, these are construction and control problems, and even though not exactly within the scope of this paper, the writer's remarks in another paper[45] may be repeated here, namely, (1) the engineer should have a thorough knowledge of the functions of the constituent parts of an hydraulic-fill dam under construction; (2) constant watch should be exercised both at the borrow-pits and at the dam to see that the quality of the structure is not affected; (3) the coarse material in the core section can be moved out and spread over the beach areas by means of drag-lines; the material which the drag-line cannot reach can be agitated and dispersed among the fines of the core section by means of jetting apparatus shown in Fig. 27; (4) in order to insure a good bond between the hydraulic-fill and the sides of the gorge particular care was taken, especially within the core area, to remove all surface cover; (5) it was soon discovered that if the surface cover within the core section was sluiced into the core pool, the fines would disperse all over the core area and leave the coarse material next to the sides of the gorge, creating blind drains; all such coarse deposits next to the sides were removed and, later, all the excavation was carried from the core area to the beaches; and (6) it is important to secure the required degree of water-tightness in the foundation both at the bottom and on the sides of the dam by cut-off trenches, cut-off walls, and grouting.

Answering Mr. Justin's question in regard to the costs of the tests and control, it should be stated that the research at Cobble Mountain, both in the laboratory and in the field, was undertaken with the full advantage of the practical and theoretical knowledge of several authorities on the subject. Even then, not enough definite information for all the necessary tests could be obtained. From the beginning to the end, no method dealing with the problem at hand, developed elsewhere, or contemplated by others, was either adopted or discarded until it was given a satisfactory trial. Members of the Laboratory Staff were sent to Columbia University and Massachusetts Institute of Technology to get acquainted with the research being done by others along similar lines. Almost all the important hydraulic-fill dams under construction in the United States were visited and problems and methods of testing were studied in each case. Several sets of apparatus were purchased and used for tests, but their cost as well as the time lost in making the experiments produced nothing of value except the knowledge that the tests were not suitable for the purpose. The cost of some of the testing machines developed on the job was only a fraction of the total expense incurred for the necessary preliminary investigations leading to the development of the final design and construction. Due to delay in the construction, because of re-letting the job to other than the original contractor for the completion of the dam, the available extra time was used to advantage for field and laboratory research, which would have been impossible had the construction progressed continuously. Due attention was paid to field control which is no less, if not more, important than laboratory control. The cost of the equipment was more than $5 000. There is no doubt that the control practiced at Cobble Mountain Dam was elaborate, but with reason; and it was certainly expensive if the cost of the aforementioned factors is taken into consideration.

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[45] "Construction of the Cobble Mountain Dam," by Harry H. Hatch, M. Am. Soc. C. E., *Journal*, New England Water Works Assoc., Vol. XLVI, No. 4, pp. 327–344.

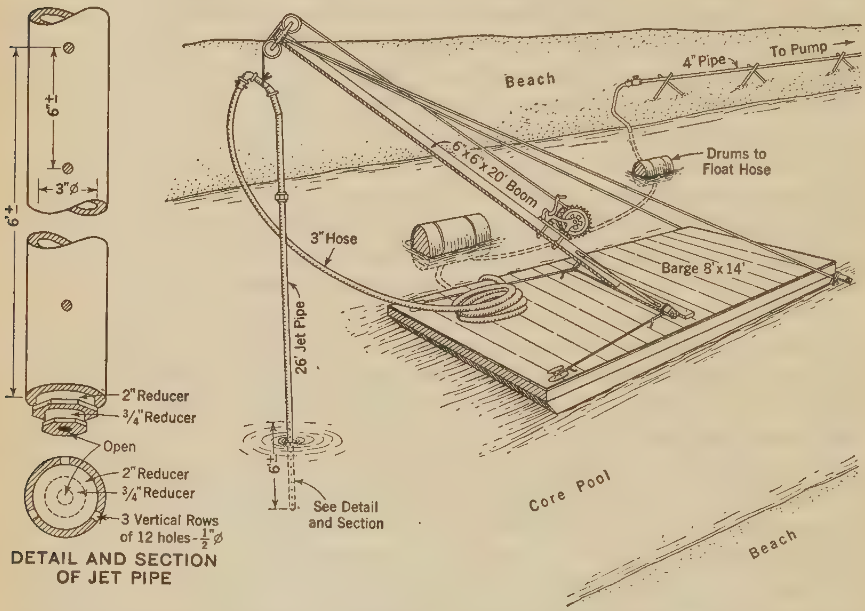
6"±
3"ø
6'±
2" Reducer
3/4" Reducer
Open
2" Reducer
3/4" Reducer
3 Vertical Rows of 12 holes-1/2"ø
DETAIL AND SECTION OF JET PIPE
Beach
4" Pipe
To Pump
Drums to Float Hose
6"x6"x20' Boom
3" Hose
Barge 8'x14'
26' Jet Pipe
6'±
See Detail and Section
Core Pool
Beach

FIG. 27.—JETTING EQUIPMENT TO DISPERSE LAYERS OF COARSE MATERIAL IN THE CORE DURING CONSTRUCTION.

A maximum number of three technicians was employed at the Cobble Mountain Laboratory. A fair estimate of cost for only laboratory control at Cobble Mountain Dam, without considering the training of the men, preliminary investigations, and experiments, would be about $15 000, or, say, nearly 1 cent per cu yd of hydraulic fill in the dam.

The samples showing the percentage of voids of the core (Fig. 2) were taken at about 15 ft below the silt or mud elevation and about from 15 days to 1 month after the material had been placed. The core material deposited in the bottom of the pool had about 60% voids.

There is a constant change in the percentage of voids in the core at any one elevation, due to the increased weight as the construction progresses. Without undue expense, the percentage of voids of the core material at various elevations at the completion of the job cannot be stated exactly; but an average approximate value from observations during construction can be selected similar to that following Equation (30) of the paper, where the assumption is that the percentage of voids in the core will vary from 50% on top to 45% at the bottom, and that the approximate average is about 47.5 per cent. These values, however, will not necessarily apply to other cores.

Under a given load for a given material there is a limiting ultimate percentage of voids that remains almost constant, as expressed by Equation (24) for the local material, keeping in mind the conditions noted in the paper.[46]

With an assumed average percentage of voids and with a given specific gravity of solid material, the acting force, L, in Equation (24) at any one elevation below the top of the dam can be computed by means of graphs similar to Fig. 28. With the assumed and final percentage of voids, the approximate time necessary for ultimate consolidation can be computed by means of Equation (30) similarly to the numerical problem immediately following that formula.

Mr. Justin's reference to Mr. Hazen's investigation of the Calaveras Dam evidently applies to the latter's statement:[47]

> "Without attempting precision of statement, it is the writer's judgment that if core material like that at Calaveras could be consolidated to 40% of voids, it would be sufficiently stable to do its share toward resisting the pressures that come on the dam without tending to disruption. Perhaps this degree of stability would be reached before consolidation had proceeded as far as 40% of voids. There is a middle ground of uncertainty. Material with 50% of voids is still unstable; and between this limit and 40% one cannot be certain just where to draw the line."

Mr. Hazen did not state, nor does Mr. Justin state, at what elevation in the Calaveras Dam, or under what given load, the 40% of voids is desirable; nor are there any data to show the ultimate rate of consolidation under various loadings, or at various heights of the dam, for the Calaveras core material.

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[46] See p. 240.

[47] "Hydraulic-Fill Dams," by the late Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 1720.

The twenty-eight core samples tested at Cobble Mountain under a pressure of 200 lb per sq in. showed a wide range in percentage of voids at ultimate consolidation. The minimum was 30% and the maximum, 41.5%, depending on particle sizes and other characteristics of the same materials. The percentage of voids at ultimate consolidation cannot be uniform throughout the core, since naturally it is a minimum at the bottom and increases gradually, depending on the uniformity of the material, toward the top of the dam.

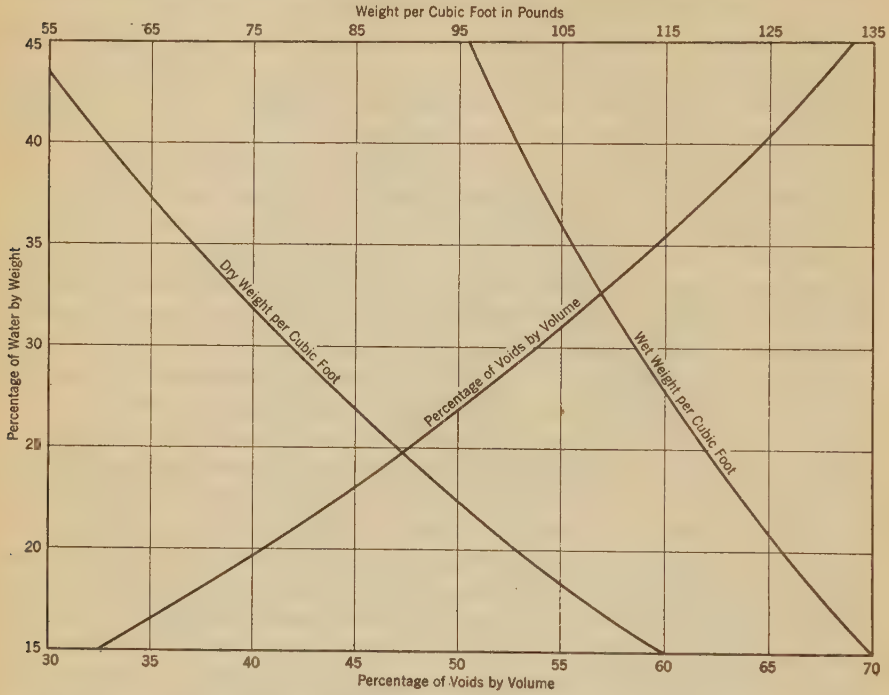

FIG. 28.—CURVES SHOWING RELATION OF WATER CONTENT TO PERCENTAGE OF VOIDS, AND WEIGHT PER CUBIC FOOT FOR SATURATED MATERIALS. SPECIFIC GRAVITY OF SOIL PARTICLES = 2.72.

The local material for which Equation (24) was obtained, has an average of about 38% of voids at ultimate consolidation under a loading of 200 lb per sq in., corresponding to the maximum pressure at Cobble Mountain Dam. At a pressure of 100 lb the same material will have 40%, at 50 lb, about 41.7%, and at 25 lb, about 43.5% of voids for ultimate consolidation. There is every reason to believe that the percentage-of-voids curve for the core, as obtained from Equation (24), is too low, and that actual consolidation in the dam can reasonably be assumed at a percentage of voids about 10% higher. The handling or the puddling of the sample while being placed within the compression apparatus produces a somewhat different arrangement of the particles, resulting in a lower percentage of voids in the material. This, according to the reasons already given in the paper under the heading, "Consolidation of the Core of Cobble Mountain Dam," will result in a lower final percentage of voids. The initial percentage of voids of the local material in question, before application of any load in the testing machine, was about 53%, whereas the initial percentage of voids of the same material in the bottom of the core pool is about 60 per cent.

Undoubtedly, it is desirable that, soon after construction, the core material shall have been almost completely consolidated under various loadings, at different elevations in the structure.

The statement in the paper that "there is no such general formula giving the seepage through any hydraulic-fill dam" similar to that proposed, was made with full knowledge of the formulas previously given by Mr. Justin "for seepage through earth dams and also for seepage through the foundation soil." Mr. Justin's formulas, which are adaptations of the Hazen and Slichter formulas, are cumbersome, are not general, and cannot be applied to the core materials under consideration.

The Hazen formula (Equation (22)), for the filter sands cannot be used for core materials. Without going into detail, briefly it may be stated that Mr. Hazen's constant varying from 400 to 1 200, applicable though it may be for filter sands, cannot be applied to core materials. The Hazen formula does not have any factor to take care of the variation in the percentage of voids of the material. It may be claimed that filter sands are more or less alike, and perhaps that they have a uniform percentage of voids; but the same argument cannot be applied to the core materials of hydraulic-fill dams in which there is such a variation in the percentage of voids. Furthermore, the effective size and the 60% size of filter sands cannot be compared with the respective sizes of core materials in hydraulic-fill dams. Here, again, the conditions are different.

Professor Slichter refers to his own formula for determining the flow of water through a column of sand,[48] but he does not state how he devised it. In the absence of any definite information, and in view of the fact that his discussion refers to sands and to the Hazen formula, the writer believes that Professor Slichter's formula will not apply to core materials.

The formula may be valid for sands within the range of sizes used in the Hazen experiments, but it is doubtful whether it will be reliable for fine particles of about 0.01 mm, as given in his tabulation of "transmission constants."[49] His results are probably due to theoretical computations, and his experiments, if any, did not cover the lower ranges given in his table. It is doubtful whether, about 1900, gradations were made as fine as 0.01 mm. The "transmission constants" tabulated by Professor Slichter, indicate that the value of K varies approximately as $V^{3.3}$, whereas Cobble Mountain experiments for core materials (Fig. 15), show that K varied as $V^{5.40}$ to $V^{7.2}$, or approximately as V^{6}, which is the average value adopted in Equation (23).

In the preceding three paragraphs will be found the reasons for the differences in Equations (53) and (54), as pointed out by Mr. Creager.

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[48] *Water Supply Paper No. 67,* U. S. Geological Survey, p. 24.

[49] *Loc. cit., No. 140,* U. S. Geological Survey.

Mr. Wade's discussion of the theory of the hydrometer is a valuable contribution on the subject. It would be desirable to have complete knowledge of the underlying principles of the hydrometer; to know exactly what is taking place during the test; why the particles do not settle according to theoretical calculations; the effect of the Brownian movement; whether there is any vortex motion; and what the real reasons are for the variation in results obtained by the hydrometer compared with those by more accurate methods. These factors are not well known yet, and any possible attempt to discuss them theoretically would have been outside the scope of the paper. More space has been given to the hydrometer than is warranted, when it is considered that the hydrometer method is only one among several for making gradation tests, and that the latter is only one of the many subjects treated in the paper.

Certain principles were given to explain reasons for the method of conducting the tests, and certain probable causes were enumerated for some of the variations of the obtained results compared with those by other more accurate methods.

The adoption of the hydrometer method, with reasonably accurate results, as described, will depend perhaps more on the simplicity and almost negligible cost of the instrument, and the speed of the tests, rather than on an immediate thorough knowledge of the underlying theory.

Quite properly, Mr. Wade points out the possibility of confusion in interpreting the statement pertaining to concentration at the center of buoyancy. This would have been clear perhaps if it had been stated that the resultant buoyant forces of the uneven concentrations at various elevations in the suspension acting on the hydrometer are observed arbitrarily at its center of buoyancy.

The assumption that the center of buoyancy is the critical point to which to measure the depth below the water surface, seemed logical and gave satisfactory results, even though there was "a slight error in the assumption," according to Mr. Wade.

By checking the readings of the hydrometer with various predetermined quantities of grammes of soil per liter of water, or with solutions of known concentration, as recommended under the heading "The Hydrometer," Mr. Wade's problem with respect to variation in the specific gravity of the material in suspension can be avoided. It is doubtful whether there will be any change in the specific gravity of a core material, during the process of settlement, provided this material is nearly free of organic matter, and provided the particles of soil in the sample are of a practically uniform specific gravity.

The dispersing agents used at Cobble Mountain were mostly ammonia, sodium oxalate, and sodium silicate. The proper quantity to be used for a given soil sample can be obtained by trial beginning with a small quantity and increasing it gradually until the right proportion is reached.

The hydrometer and the thermometer readings can be estimated to the nearest one-tenth, and no doubt the nearest one-half gramme or one-half degree would suffice. Even though the corrections made on the hydrometer readings in Table 3 could have been neglected, they were inserted to show the arrangement of observations and the method of computation.

At 68°F the viscosity of water was assumed to be 0.01005 poises, or 1.005 centipoises. It is desirable, of course, to have water in the graduate with a temperature of 67°F in order to avoid corrections entirely, which, otherwise, should be made according to Equation (10). The corrections can easily be obtained from a graph of this equation.

In addition to the gradation, Mr. Dore suggests making determinations of grain shapes, their classifications, their surface and other characteristics. This would tend toward a high degree of academic refinement; but, no doubt, such studies made in a research laboratory would be of value in construction work if and when they can be readily applied in a practical manner. It is doubtful whether they can be included among the tests to be made in the field laboratory during construction. It should be remembered that laboratory and field control must supply data for quick and decisive action because of extreme variation of the characteristics of the material on hand and the urgent necessity of speed.

The surface characteristics of the grains, and their hardness are of great importance, and periodic examinations by microscope and otherwise were run at Cobble Mountain, with the result that important changes were made during construction. It is obvious that the information will be more complete if various characteristics and properties of the material in question could be presented in an accepted and compact form. It is to be hoped that investigation of the coefficient of friction of earth dam materials will continue until sufficient data are obtained on this subject.

The merits and the limitations of the three methods of testing for percolation, as finally adopted at Cobble Mountain Dam, were stated in the paper. Each has its own advantages, depending on the available time for tests and consideration of cost. It was fortunate that at Cobble Mountain each of these methods was available and was used, because some of the results obtained and conclusions reached are due to their collective use.

In regard to the vertical and horizontal capillary tube methods, it may be stated that the dry sample of material placed in a glass tube cannot be converted into a material similar to that in the core of a dam. The dry sample often contains small lumps of material that are not broken up even with reasonable care in its preparation. The lump content of samples is not the same, and it does not follow any rule. The presence of lumps accelerates the capillary rise of water in the sample in proportion to their size and number.

These methods are more or less approximate. The vertical capillary method requires a series of master curves, prepared carefully for similar materials, and by elaborate methods. The horizontal capillary formula contains a constant, depending on the percentage of voids and the character of soils, which can only be obtained after the coefficient of permeability of that one material has been determined carefully by numerous experiments conducted with reliable methods. The fact that similar approximate values or results could be obtained more easily and readily from Equation (23) was the reason for discarding them without prejudicing their value for testing rolled earth dam material.

It was not intended that Equation (23) should replace the actual tests and direct measurement of the coefficient of permeability of the core material, but that it should be used only for estimating, approximately, the coefficient of permeability. This formula is the best tool available for the purpose. In order to be of more general use in its application to the wide range of core materials, it requires a factor, expressed in terms of the area, perhaps below the curve, between size limits of 60 and 10%, or (which is better) as nearly as possible between the full limits of 100 and 0 per cent.

The relation between the coefficient of permeability, K, and the effective size, d, as shown in Equation (23), was obtained from the results of thirty-nine complete experiments using two different sets of percolation apparatus. A study of the plottings of Fig. 29 will show that for the large percolator the range of the coefficients is from 84 to 480 and the general average for the 18 tests is 176. The compression apparatus gives a range of coefficients from 30 to 311, or a general average of 106 for 21 tests. The average of the values obtained from the two sets of apparatus is 141.

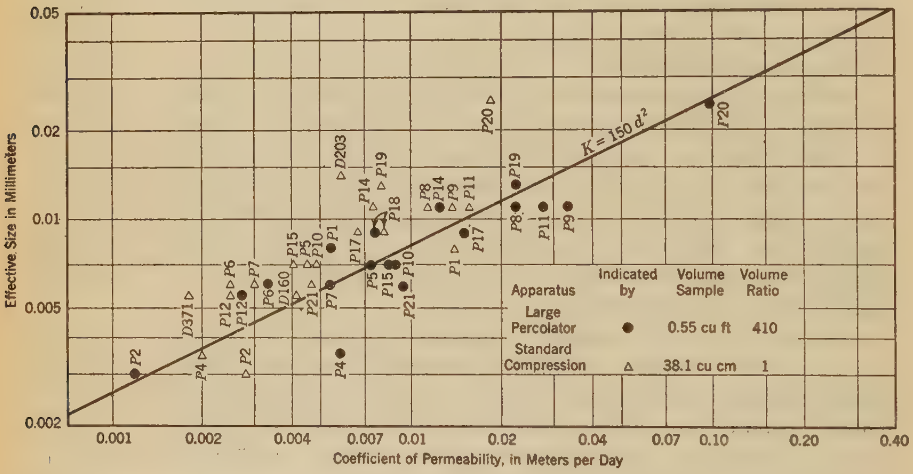

FIG. 29.—RELATION BETWEEN EFFECTIVE SIZE AND COEFFICIENT OF PERMEABILITY AT 45% VOIDS AND 50 DEGREES FAHRENHEIT.

The following test results were questioned during the experiments:

Sample No.	Value of K	Apparatus used
$P1$	228	Standard compression
$P2$	311	Standard compression
$P4$	480	Large percolator

Omitting these questionable data, the range of the coefficients is from 84 to 278 for the large percolator and the general average for the reliable 17 tests is 157. Similarly, the range of coefficients is from 30 to 163, for the compression apparatus, or a general average of 89 for 19 tests. The average of the values obtained from the two results is 123.

The adoption of *K* equals 150 instead of 141 or 123 is due to preference given the large percolator, which may be more reliable for this test in view of the fact that its sample was more than 400 times larger than that used for the compression test. Equations (23), (30), and (43) afford valuable information if used with discretion, keeping in mind their restrictions or limits as pointed out in the paper.

Due to the fact that the gradation results obtained by the hydrometer and elutriator methods are almost alike, Professor Krynine raises the question as to why the elutriator needs to be used for future tests of the same character. At Cobble Mountain, hydrometer results were often checked by the elutriator, thus making certain that the hydrometer results were reasonably accurate. If the hydrometer runs at a construction laboratory can be checked periodically by a more accurate method, whether the elutriator or some other, there is no reason to invest money in the apparatus and devote time to making elutriator tests.

Attention is called to some facts in regard to the arbitrary assumption relating to the center of buoyancy of the hydrometer bulb, which, when fully studied and analyzed, may clarify further the reasons of the discrepancy (especially in the lower limits), between the results of the hydrometer and a more accurate method.

The curves in Fig. 10 were plotted only within the limits of actual experiments, and the corresponding equations are likewise limited. It seems logical, however, that as the vertical pressure approaches zero the coefficient of friction should approach infinity.

It is regretted that Professor Krynine did not reproduce from German and Russian books on seepage, a formula, if any, similar to Equation (43), that would give the seepage through a hydraulic-fill dam. It would be interesting to compare such a formula with Equation (43) of this paper. Equation (43) was derived without any knowledge of the works referred to by Professor Krynine.

Mr. O'Shaughnessy states that "the core material is of particularly thin section for this type of dam." This is true for Cobble Mountain Dam as compared with others in existence. The thin core section has given good satisfaction, however; the reservoir has been full for some time and the seepage through the structure has been negligible.

The large weep-holes in the toe dam will scarcely affect the resisting strength of the structure, and there is no danger of their being plugged by fine material, resulting in hydrostatic pressure back of the toe dam.

According to Mr. Maitland C. Dease, to whose report Mr. Cox refers, considerable work has been done at the Alexander Dam along lines suggested by Professor Terzaghi for tests on clays. These methods were not given much consideration at Cobble Mountain, because the problem was not concerned with clays. The "scope and intent" of the testing program at Cobble Mountain Dam was practical, and was as complete as possible.

Highly theoretical tests made in research laboratories, are valuable, but in the construction of an hydraulic-fill dam the most important criterion is effective field control, guided by laboratory tests. The laboratory investigations for this purpose should bear immediate results, which, in turn, should become useful for immediate application. The interpretation of the laboratory and field investigations is of utmost importance. Neither elaborate machinery, long-period tests, nor complicated equations are necessary in order to detect danger signals pertaining to the safety of the structure. Simple terms, common sense, and practical judgment in interpreting tests are more valuable than tests requiring months, which produce complex equations not readily understood. Lack of practical interpretation often leads to disastrous results, no matter what the testing program may have been.

The aim at Cobble Mountain was to simplify the testing methods and to make frequent tests so that their results could serve as guides during the construction. Changes were possible immediately, without hindrance to the progress of construction, and yet with time to produce the required stability in the structure.

A study of Mr. Dease's full report reveals no information in regard to the number of tests for the coefficient of friction at Alexander Dam comparable with about 550 tests made at Cobble Mountain Dam. The report indicates that during a month only six core samples and only three beach samples were tested, whereas at Cobble Mountain Dam an average of more than 100 core samples alone were tested each month. There is no evidence in the report to substantiate Mr. Cox's statement that the testing program at Alexander Dam "was considerably more extensive than that at Cobble Mountain, or than has heretofore been attempted in connection with hydraulic-fill construction."

Shearing resistance of beach material is of interest; no studies of the effects of cohesion and friction in their combined relation to shear were needed at Cobble Mountain, because the beaches were composed of almost cohesionless sand and gravel.

Whereas a material as fine as that used in the Alexander Dam might require the pycnometer method for accurate determinations of the specific gravity, the simple displacement method gave entirely satisfactory results with the coarser and less cohesive material used in the core of the Cobble Mountain Dam. On the other hand, considering that at Alexander Dam "the specific gravity of the separate grains of a representative sample varied from 5.2 to 2.2" (a variation of 236%), it is questionable if it was necessary to use such refined methods of determining the specific gravity, because the "simple displacement methods were frequently in error as much as 15 per cent."

The voids in the large percolator described in the paper were obtained with sufficient accuracy by simple moisture content determinations, and the variation within the apparatus was measured and found to be insignificant.

The stand-pipe permeameter is admittedly difficult to load with some types of material and under some conditions and hence it is limited in its application, but where it is suitable, it is simple and quick.

The term, "percentage of voids," is preferred to that of "voids-ratio," because of its simplicity, and because of the fact that it has been used for many years and is readily understood by practising engineers.

The simplicity of Equations (21) and (24) compared with Equations (46) and (47), speaks for itself especially when "there is little actual difference over a moderate range of voids-ratio." Both sets of formulas are empirical, and are applicable to particular materials. The ultimate aim might be to develop a set of formulas sufficiently general and complete to cover all materials and containing commonly accepted terms. Until such formulas are available the most simple, rather than the most technically precise, formulas would seem desirable for working purposes.

A study of the curves in Fig. 22 shows that they can be expressed by the general formula given in Equation (21), but that the exponent, m, will be much higher than 7.2 as given in Table 5.

The permeability coefficients in Table 5, evidently do not take into consideration the temperature factor, which is important.

The permeability coefficient given for the Cobble Mountain Dam material is not correct, and, furthermore, the units of centimeter per minute and meter per day are not the same. Chemical content and mineralogy were not serious problems at Cobble Mountain Dam, but their importance as pointed out seems evident, and of considerable interest.

The time required for reasonably accurate work by microscopic examination makes this more properly a research, rather than a field laboratory, test. Due to the relatively uniform specific gravity of Cobble Mountain Dam material the difficulty with variations was not encountered; there was no indication of excessive hydration. The other factors are undoubtedly present and their influence depends on the character of the material. These factors must be estimated for material from any given locality, since they are not susceptible at present to complete, or exact, analysis. The discrepancy in the gradation curves obtained by hydrometer and microscope, as shown in Fig. 21, is interesting, if these curves are the results of numerous sets of experiments.

The discussion of shearing tests on clays and clay-like soils is of considerable interest and possible value in soil studies. Its importance with direct relation to an hydraulic-fill dam is not so evident. The fundamental concept of a true hydraulic-fill dam is one in which the beach section is of a sufficiently coarse material to permit almost immediate drainage and consolidation of itself, such as to allow suitable drainage facilities for the core. The safety of an hydraulic-fill dam being dependent on the coefficient of friction, or shearing resistance of the beach section, the application of shearing tests to this coarser material would seem more important and applicable.

Fig. 30 (in which the curves are similar to those of twenty-six other sets of consolidation experiments made at Cobble Mountain) and Table 6 will substantiate the statement that "the rate of consolidation of material under a given constant load, shortly after the application of the load, becomes almost uniform." For example, Table 6 shows that 100% consolidation was reached for Curve *A* in 20 hr, whereas, by Fig. 30, the sample had 97% consolidation in 10 min. Likewise, Curve *B* reached 100% consolidation in 19 hr (Table 6), whereas the sample material reached 80% consolidation in 10 min. This rapid consolidation undoubtedly is due to the physical characteristics of the Cobble Mountain material. Perhaps the conditions would be different for clays, but the same tendency is evident in Fig. 31, which was replotted from Fig. 23 for Alexander Dam material. In the absence of any definite data in regard to the initial and final voids of the sample material, assume that they conform to the limits of the plotting in Fig. 31; that is, assume that the percentage of voids was 63.065% when the load was first applied and 62.767%, after the material had become completely consolidated. The consolidation is very rapid at the beginning, and for almost the last half of the total time required for the ultimate consolidation of this material there is no reason why the rate of consolidation could not be assumed as "almost uniform."

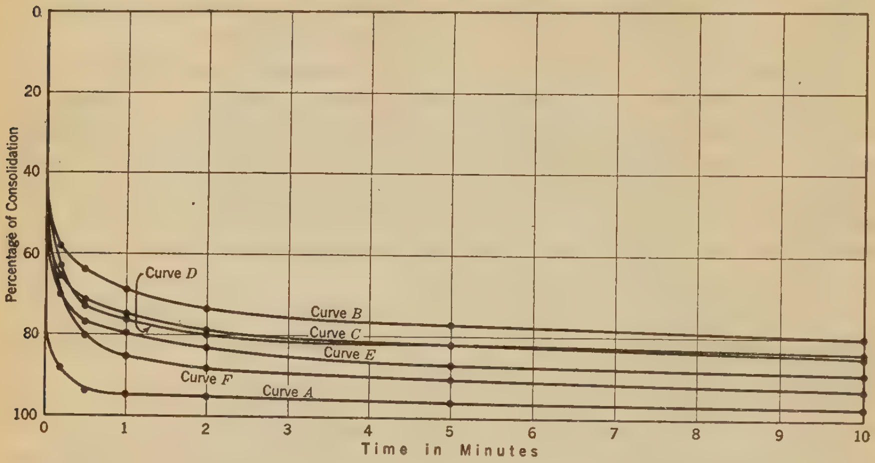

FIG. 30.—CURVES SHOWING RATE OF CONSOLIDATION OF A COBBLE MOUNTAIN DAM CORE SAMPLE.

TABLE 6.—IDENTITY AND DESCRIPTION OF CURVES IN FIG. 30

Curve in Fig. 30	Pressure, in pounds per square inch	Percentage of Voids		Time, in hours, for 100% consolidation
		Initial	Final	
A	8.6	49.6	45.0	20
B	13.0	45.0	44.4	19
C	25.0	44.4	43.2	23
D	50.7	43.2	41.6	21
E	101.5	41.6	39.6	17
F	199.5	39.6	37.6	22

It was clearly stated that the consolidation study was only "an attempt * * * to estimate roughly the time necessary for an approximate ultimate consolidation." So many factors enter into the computation—there are so many assumptions and uncertainties—that it is not worth while to go into too much refinement. The purpose of the study was to determine whether the approximate consolidation would take, say, 2 or 20 years; even if the results obtained for the approximate consolidation vary from those of the actual, say, 100%, or, from 2 to 4 years, the study serves the purpose for which it was made.

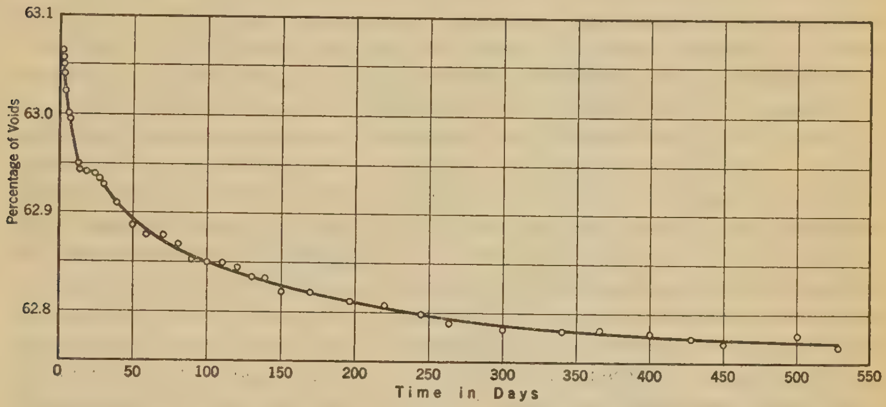

FIG. 31.—COMPRESSION UNDER A STEADY APPLICATION OF LOAD (RE-PLOTTED FROM FIG. 23).

Mr. Cox has computed the time for the consolidation of the Cobble Mountain Dam by Professor Gilboy's method, and the results do not agree with those of Equation (30); but the difference between the results by the two methods is due only to errors in Mr. Cox's computation, or assumed dimensions, rather than to the assumptions, or to the derivation of Equation (30).

"Taking an elliptical cylinder (semi-major axis, 5 930 cm, and semi-minor axis, 4 780 cm) that has an area roughly equivalent to that of the dam core, and a ratio of diameters about that of height to one-half the width of the actual core," as recommended by Mr. Cox, is erroneous and cannot apply to the core of the Cobble Mountain Dam. Far from being "roughly equivalent," the area of an elliptical cylinder with the given dimensions is approximately five times that of the dam core (see Fig. 17). In order to have "a ratio of diameters about that of height to one-half the width of the actual core," the semi-minor axis should have been about one-half the value assumed by Mr. Cox. Applying the dimensions used by Mr. Cox to the Cobble Mountain Dam, or assuming twice the width of the actual core, and without going into any further details, it is seen, in Equation (30), that the time of consolidation varies as the square of the core width; or, in this case, it would be increased four times that given in connection with Equation (30) in the original paper which agrees with the tabular values given by Mr. Cox. It is interesting to note that results by Equation (30) agree so well with those obtained by more complicated and elaborate methods.

The seven reasons given to warn against deceptive conclusions, were pointed out after serious and repeated practical observations. They are still valid, at least for the Cobble Mountain materials, notwithstanding the objections offered by Mr. Cox.

It is not possible, with a few cubic centimeters of material enclosed in a polished brass cylinder, to duplicate the changing conditions and the characteristics of 300 000 cu yd of core material in a dam 263 ft high. There is a practical limit to the possibility of determining the reactions in a large core by means of theoretical laboratory investigations conducted under ideal conditions and with small test samples that are not necessarily representative.

Undoubtedly, much additional research will have to be done, and perhaps new tests and testing methods will be devised in connection with hydraulic-fill construction; but the time has come when engineers connected with this type of construction should get together and co-ordinate the various methods of successful testing and to classify data gathered from successful structures. It is time that engineers should talk the same language and express themselves in the same terms and definitions so that differences in results can be distinguished readily by practising engineers who are not concerned with abstruse theories, the application of which to the problem on hand may yet be somewhat questionable. There is no reason why, until something better is offered, the consensus of opinions and facts compiled by actual experience should not serve as a guide for this type of construction. Establishment of standard tests and testing methods in no way implies that further research should be stopped; on the contrary, the present chaotic condition will be clarified, and much duplication of work will be avoided. Time thus saved can be used for research in the other investigations necessary to a solution of the many features of the problem.

Throughout the paper readers were cautioned against accepting any conclusion as final, and against applying it to another locality in which the materials were different. For a quick decision, or for estimating the probable results during construction, the utility of data obtained from grain sizes and their gradation is paramount until some better and quicker method can be developed. With thousands of yards of material daily going into the structure, there must be some practical means by which the engineer responsible for its safety can readily determine the course to be followed in the control of the construction, if conditions so require. It would be out of the question to stop the construction for weeks at a time in order to determine what should be done.

The writer feels grateful to the discussers, even though some of the views expressed were contrary to those in the paper. After all, however, the difference in opinion will help to bring out the fundamental principles of the subject-matter more definitely than ready acquiescence.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 1863

FOUNDATION TREATMENT AT RODRIGUEZ DAM[1]

By Charles P. Williams,[2] M. Am. Soc. C. E.

With Discussion by Messrs. L. A. Robb, Francisco Gomez-Perez, F. A. Noetzli, and Charles P. Williams

Synopsis

This is an account of the design and construction of the foundation structure for the Rodriguez Dam, on the Tijuana River, Baja California, Mexico. The dam is of the Ambursen type, and, as designed, has a greater height than any dam of that type heretofore built. The bed-rock in parts of the stream bed is of inferior quality, and, in some parts, disintegrated. A geologic fault about 20 ft. in width extends along the border of the stream bed.

A brief introductory description of the project and of the dam is given, the geologic formation at the site is described, and the foundation structure that supports that part of the dam which otherwise would rest on poor foundation, is described and illustrated.

Introduction

The Rodriguez Dam is situated on the Tijuana River, in the northern part of Baja California, Mexico, about 11 miles easterly from the Town of Tijuana. Its purpose is the storage and diversion of water for the irrigation of about 5 000 acres in the Tijuana Valley in Mexico, and for domestic supply for the municipality of Tijuana which has a population of 10 000.

The drainage basin of the river, which lies partly in Baja California and partly in San Diego County, California, has an area of 1 670 sq. miles. This is greater than that of any other stream south of Los Angeles, Calif., and west of the Colorado River; it nearly equals the total area of the drainage basins of all other streams in San Diego County. The stream has two principal branches, upon the larger and more southerly of which the Rodriguez Dam has been located. The area of the drainage basin above the Rodriguez site is 938 sq. miles.

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[1] Presented at the meeting of the Irrigation Division, Sacramento, Calif., April 24, 1930.

[2] Cons. Engr., Los Angeles, Calif.

Like all streams in the vicinity, the flow of the Tijuana River in ordinary years is very small, and the supply is necessarily dependent upon the storage of flood waters. Periods of from 5 to 7 years, in which the flow is insignificant, may be expected, and consequently large carry-over storage is important. In order to furnish a sufficient irrigation and municipal supply from Rodriguez Reservoir, a capacity of about 110 000 acre-ft. is necessary, requiring a maximum reservoir water surface about 180 ft. above the stream bed.

While streams in this vicinity, even during the rainy season, have very small flows in ordinary years, excessive floods sometimes occur. The largest flood of definite record was that of 1916. Some of the streams in San Diego County, at the peak of that flood, had flows corresponding to coefficients ranging from 5 to 7 in Talbot's formula. At that time, the flow of the Tijuana River, below the Arroyo Matanuco, which empties into the river immediately below the Rodriguez Dam site, and which has a drainage area of only 115 sq. miles, was about 70 000 sec-ft. It is estimated, however, that a flood of 150 000 sec-ft. is possible.

The Rodriguez site is in a gorge which, at its narrowest section, has a width at stream bed of about 100 ft.; at 130 ft. above stream bed, the width is about 750 ft. Beyond this, on either side, the gorge is flanked by a saddle and by gentle slopes which, immediately adjacent to the site, rise to 200 ft., or more, above stream bed.

Rock outcrops throughout a considerable portion of the site, the rock of the canyon walls at the gorge, that of the western slope, and that of a part of the eastern slope being rhyolite, while that of the easterly part of the eastern slope is granite. The rock is fused at the contact between the granite and the rhyolite, showing the latter to be the older rock, and indicating that no great leakage from the reservoir may be expected along the contact. The surface rock of the canyon walls, in the narrowest section, is fresh and hard, although considerably broken; that of the slopes is generally badly weathered, and, in some parts, considerably disintegrated. On either side of the canyon, are a number of cleavage planes which dip toward the stream bed and, in some cases, up stream.

PRELIMINARY INVESTIGATIONS

In the preliminary investigation of the dam site, nine borings were made in the stream bed and fifty-two test pits were excavated on the side hills. The latter indicated that suitable rock for foundation could be secured, ordinarily, with comparatively shallow excavation. At the site of the east wing of the dam, for a distance of about 500 ft., suitable rock was found at depths ranging from 15 to 25 ft., while at the site of the proposed spillway, on the west side of the river, the depth for about 100 ft. was from 15 to 40 ft. Five of the nine borings in the stream bed indicated satisfactory rock at depths not exceeding 50 ft. From four of the borings, no cores could be obtained, the borings being made easily with a chopping bit. The materials obtained from two of these four borings were clean sharp stone chips, but the materials from the other two gave considerable evidence of disintegration.

After the borings had been made and a number of the test pits had been excavated, the geological formation at the site and in the vicinity, both in Mexico and in the United States, was examined by a geologist, who reported that, in his opinion, the rock in the stream bed, that of the western flank, and that of a part of the eastern flank, would be found suitable for a foundation for a concrete dam, but that he considered an earth and rock-fill dam to be best suited for the saddle on the eastern flank.

Brief Description of the Dam

The dam is of the Ambursen type. Its maximum planned height is 187 ft. above the stream bed and about 240 ft. above the lowest foundation bed-rock. The length of the crest is about 2 000 ft. For the central part of the dam and for the west wing, the spacing of the buttresses is 22 ft., center to center. The buttresses for the central part and the west wing vary in thickness from 19 in. at their thinnest section, to 66 in. at 200 ft. below the crest. The thickness of the deck varies from 25 in. at 15 ft. below the crest to 64.5 in. at its lowest portion, 192 ft. below the crest. The lower face of the deck has a slope of 1 on 1, and the down-stream face of the buttresses a batter of 5 on 1.

In each of three of the stream-bed bays of the dam, the plans provide for a 5 by 5-ft. sliding sluice-gate, operated by a hydraulic cylinder. The service outlet works consist of two 30-in. cast-iron pipes, at an elevation about 33 ft. above the stream bed, in each of which will be installed one 30 by 24-in. needle-valve, and two 30-in. emergency gate-valves. The discharge will be measured by a Venturi meter.

The spillway in the west wing of the dam is to be controlled by nine 30 by 30-ft. structural steel gates on caterpillar bearings. With an elevation of the water surface 6.5 ft. below the crest of the dam, the spillway is designed for an estimated capacity of 150 000 sec-ft.

The Foundation

On February 17, 1928, Gen. Abelardo L. Rodriguez, Governor of the Northern District of Baja California, entered into a contract with the Ambursen Dam Company for the design and construction of a dam at the Rodriguez site, to be subject to the approval of the Government.

Excavation in the stream bed was begun by sub-contractors on May 21, 1928. In August, bed-rock was encountered at several points near the up-stream limit of the excavation, at about 35 ft. below the original elevation of the stream bed. The rock was much broken and not suitable for buttress foundations. A geologic fault, about 20 ft. in width in the exposed portion, which extended in a direction nearly parallel to the stream bed along its eastern margin, and which had not been indicated by surface conditions, was revealed by the excavation. The excavation for the cut-off wall, begun in August, disclosed badly disintegrated rock in mid-stream.

Early in September, Dr. F. L. Ransome, Consulting Geologist, Dr. Paul Waitz, Consulting Geologist for the National Commission of Irrigation, of Mexico, and the late A. J. Wiley, M. Am. Soc. C. E., were called for consultation regarding the conditions affecting the suitability of the site for the construction of a dam of the type proposed. The problems to be considered were the possibility of excessive leakage from the reservoir, the practicability of obtaining adequate foundation for the dam, the probable depth and extent of cut-off wall required, and the possible danger of future seismic disturbance. A further inspection was made by Messrs. Wiley and Ransome on November 11 to 13, 1928, at which time the cut-off trench had been excavated to a depth of about 33 ft. below the bed-rock surface.

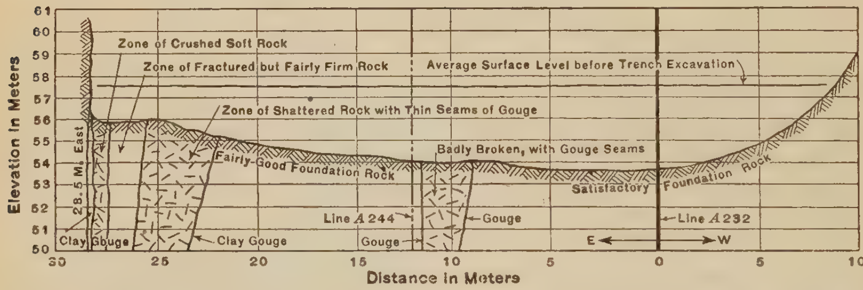
Elevation in Meters
61
60
59
58
57
56
55
54
53
52
51
50
Zone of Crushed Soft Rock
Zone of Fractured but Fairly Firm Rock
Average Surface Level before Trench Excavation
Zone of Shattered Rock with Thin Seams of Gouge
28.5 M. East
Badly Broken, with Gouge Seams
Fairly-Good Foundation Rock
Satisfactory Foundation Rock
Line A 244
Gouge
Line A 232
Clay Gouge
Clay Gouge
Gouge
E
W
30
25
20
15
10
5
0
5
10
Distance in Meters

FIG. 1.—RODRIGUEZ DAM: LONGITUDINAL SECTION OF CUT-OFF TRENCH IN STREAM BED.

After this inspection, Dr. Ransome reiterated his opinion as reported previously under date of September 13, 1928. In the report mentioned it was pointed out that the site chosen was satisfactory in most respects. One grave defect, however, was the existence of a fault zone which, although it had been inactive for more than 100 years, might conceivably move in the future. Nevertheless, such movement is not expected and Dr. Ransome stated that the risk was not so great as to justify the condemnation of the site.

A longitudinal section of the cut-off trench in the stream bed, showing the formation disclosed at the time of Dr. Ransome's inspection, is shown by Fig. 1. The general character of the rock in the stream bed is shown by Fig. 2.

The bearing power of the rock in the stream bed varies greatly in different parts of the foundation area. Four of the buttresses, if founded on the natural material, would rest for a part of their lengths upon material of relatively poor bearing power. The easterly two of the four would be founded for part of their lengths upon the material of the main fault. It was realized, therefore, that it would be necessary to support the four buttresses by a structure of a type such that the loads from the weaker parts of the foundation area would be transferred to those parts having ample supporting power.

After a number of preliminary studies had been made a design was submitted providing for a reinforced concrete floor, 8 ft. thick, covering the entire stream bed within the foundation area. Above, and tied to the floor by steel reinforcement, were massive reinforced concrete ribs, 5 ft. thick and spaced 20 ft., center to center. These ribs were designed so as to transfer the buttress loads, by internal arch action, to the sound rock on either side of the stream bed. The internal arches were defined within the ribs by means of bands of reinforcing steel which were extended through the buttresses, the buttresses and ribs forming a cellular structure. Besides serving as a support for the buttresses, the ribs were intended to serve as horizontal bracing for the rock of the canyon walls, which was cut by a number of cleavage planes sloping toward the stream bed.

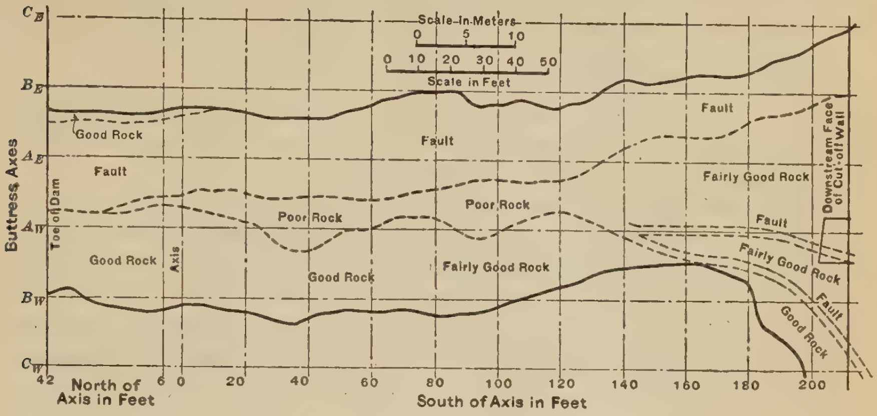
Scale in Meters
0 5 10
0 10 20 30 40 50
Scale in Feet
Buttress Axes
C_B
B_E
A_E
A_W
B_W
C_W
Good Rock
Fault
Toe of Dam
Axis
Poor Rock
Good Rock
Fault
Poor Rock
Fairly Good Rock
Fault
Fairly Good Rock
Downstream Face of Cut-off Wall
Fault
Fairly Good Rock
Fault
Good Rock
42 North of Axis in Feet 6
0 20 40 60 80 100 120 140 160 180 200
South of Axis in Feet

FIG. 2.—CHARACTER OF FOUNDATION ROCK IN RIVER CHANNEL.

The design was reviewed by the late Fred A. Noetzli, M. Am. Soc. C. E., and was found sufficient for the purpose for which it was intended. Later, it was considered by a Consulting Board consisting of Messrs. Wiley, Noetzli, and the writer, assisted by Spencer W. Stewart, M. Am. Soc. C. E., which Board concluded that while the proposed structure had been found by analysis to be reasonably sufficient, a more satisfactory and only a little more costly structure would be obtained by building a voussoir concrete arch, continuous from the up-stream cut-off wall to the toe of the dam, with a curved extrados and a flat base, resting on the natural material. It was proposed that the arch consist of unreinforced massive voussoir blocks with staggered radial joints, each block extending, without construction joints, from the base to the extrados.

In designing the structure the upper part was to be considered an arch, with the theoretical intrados intersecting the canyon walls above the lower limit of sound rock. It was to be assumed that the load of the concrete below the theoretical intrados would be carried by the natural foundation material, and that the arch would support only the buttress loading with reservoir water surface at the elevation of assumed high water, the weight of the arch above the theoretical intrados, and the water load between the buttresses to the elevation of assumed low water below the dam. Since the structure was assumed to be fully submerged at all times, stresses due to variations in temperature were to be neglected. Uplift under the structure, due to low-water pressure, was to be assumed. Further consideration of the problem led to the adoption of a definite construction joint at the intrados.

On account of the importance of the problem, independent designs were prepared by Mr. Noetzli and the contractor. These were considered by a board consisting of Messrs. Wiley and Noetzli, C. V. Davis, Assoc. M. Am. Soc. C. E., and the writer, and final conclusions were reached regarding the type of structure and important details.

The essential features of the adopted design are shown by Figs. 3 and 4. From Fig. 3 it will be observed that the four buttresses, *B* East, *A* East, *A* West, and *B* West, are supported by the arch. In order, in so far as possible, to avoid undesirable twisting moments in the arch barrel and irregular deflection along the line of support of any buttress, the axial line of the arch crown was made parallel to and equi-distant from the center lines of the buttresses, *A* East and *A* West. This made it necessary, at some sections, to use arch spans of greater length than would have been required to span the fault and the rock of inferior quality. On account of the varying widths to be spanned, and of the non-uniform loading, it was necessary to use arches of greater rise at the up-stream and down-stream parts of the structure, than in the central part, as shown by Fig. 4.

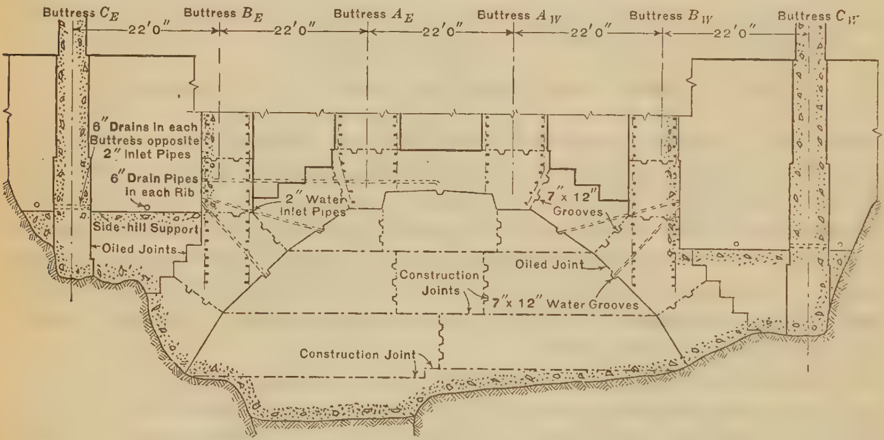
Buttress C_E
Buttress B_E
Buttress A_E
Buttress A_W
Buttress B_W
Buttress C_W
22'0"
22'0"
22'0"
22'0"
22'0"
6" Drains in each Buttress opposite 2" Inlet Pipes
6" Drain Pipes in each Rib
Side-hill Support
Oiled Joints
2" Water Inlet Pipes
7" x 12" Grooves
Oiled Joint
Construction Joints
7" x 12" Water Grooves
Construction Joint

FIG. 3.—TYPICAL SECTION OF FINAL DESIGN, SHOWING TRANSITION, RODRIGUEZ DAM.

The arch is designed to carry the full superimposed load. It is expected, however, that a considerable part of the load will be supported through the sub-intradosal concrete by the natural material in the stream bed. Where, however, such material has not supporting power sufficient for the entire load and there is incipient settlement, a part of the load will be taken by the arch, thus relieving the natural material of as much of this load as may be necessary to prevent further settlement.

Before beginning the construction of the sub-intradosal concrete blocks, cut-off trenches about 3 ft. deep, 2 ft. wide, and spaced 25 ft., center to center, were cut across the stream bed, in the fault zone, and where the rock was of inferior quality. The stream bed was then covered with a concrete mat, not less than 2 ft. in thickness. Substantial key-ways, to prevent sliding, were constructed in the upper surface of the mat.

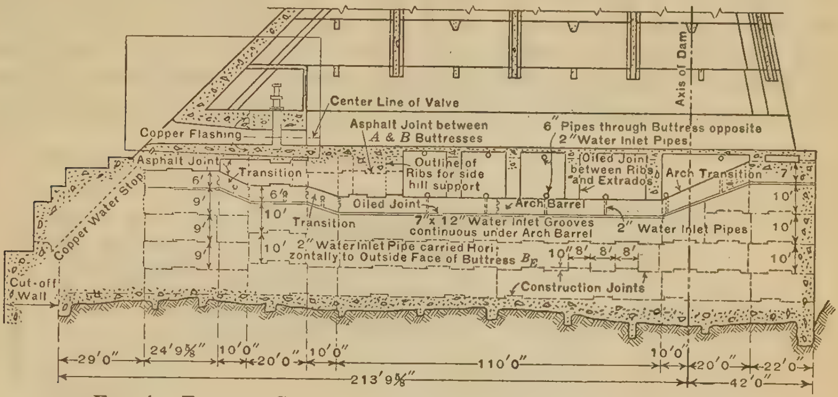
Center Line of Valve
Axis of Dam
Copper Flashing
Asphalt Joint between A & B Buttresses
6″ Pipes through Buttress opposite 2″ Water Inlet Pipes
Asphalt Joint
Transition
Outline of Ribs for side hill support
Oiled Joint between Ribs and Extrados
Arch Transition
Copper Water Stop
6′
9′
9′
9′
6′9
10′
10′
7′
10′
10′
10′
Oiled Joint
Arch Barrel
Transition
7″x 12″ Water Inlet Grooves continuous under Arch Barrel
2″ Water Inlet Pipes
2″ Water Inlet Pipe carried Horizontally to Outside Face of Buttress B_E
10″
8′
8′
8′
Construction Joints
Cut-off Wall
29′0″
24′9⅝″
10′0″
20′0″
10′0″
110′0″
10′0″
20′0″
22′0″
213′9⅝″
42′0″

FIG. 4.—TYPICAL SECTION THROUGH BUTTRESS, RODRIGUEZ DAM.

The sub-intradosal concrete was constructed of inter-locking blocks, as shown in Fig. 3. The upper surfaces of the blocks, upon which the arch barrel rests, were finished smooth and painted with oil to prevent adhesion. Transverse keys about 6 in. deep and 10 ft. wide, were constructed in the sub-intradosal blocks in order to prevent sliding of one course upon another, or sliding of the arch upon the blocks.

Normally, the water pressure under the arch will be that due to low water below the dam. During a flood, the water below the dam might rise within a short time to 15 ft., or more, above low water. This, coming at a time when the reservoir would be full, would impose a heavy load upon the arch, unless provision was made for equalizing the pressures upon the extrados and the intrados. To accomplish this, water-inlet grooves were constructed the entire length of the arch, with 2-in. inlet pipes at 15-ft. and 20-ft. intervals. It was first intended that these grooves should be filled with screened gravel, ranging in size from ¼ in. to ¾ in., the gravel to be covered by a layer of porous mortar. Owing to some of the grooves being on steeply sloping surfaces, it was found very difficult to retain this type of filling in place. The grooves were filled, therefore, with pre-cast blocks of mortar, consisting of 1 part cement to 10 parts of coarse manufactured sand. These blocks were so porous as to offer very little resistance to the passage of water. After being placed in the grooves, they were covered with a coat of mortar consisting of 1 part cement and 3 parts river sand.

Between the haunches of the arch and the walls of the canyon, concrete filling was placed to varying heights, depending upon the apparent stability of the rock walls. Above this filling and above the extrados of the arch, concrete walls 5 ft. thick, spaced generally 20 ft., center to center, have been constructed to the elevation of the stream bed, these walls, as well as the haunch filling, being for the purpose of bracing the side walls of the canyon, as a preventive of possible sliding on planes of cleavage.

In order that it may be possible at any time to determine the upward water pressure at various points at the base of the foundation structure, thirty-five 4-in., galvanized and dipped pipes have been placed in the structure, leading from the points at which the uplift pressure is to be measured, a few feet upward, then horizontally to four centralized points, from which they continue vertically upward through the structure. Eventually, the pipes will be extended vertically upward to observation platforms above high water below the dam. The lower end of each vertical riser will be always below the hydraulic grade at the point where the pressure is measured. The pressure is determined by observing the elevation of the water surface in the riser pipes, or by a pressure gauge, if, in any case, the elevation of the hydraulic grade at the point to be measured is higher than the top of the riser pipe. The plan of the pipe system is shown by Fig. 5.

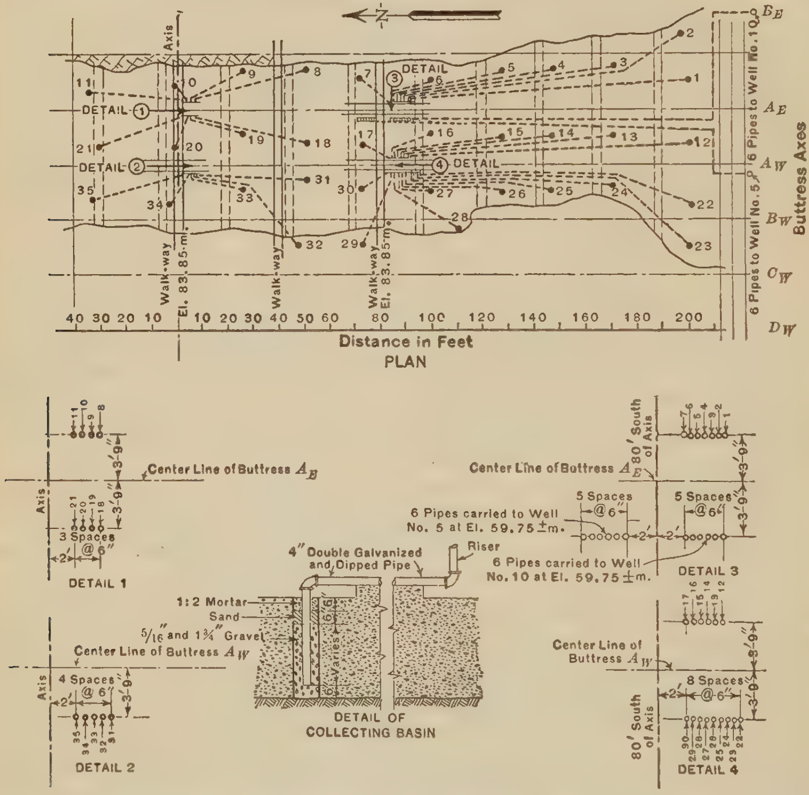
N
Axis
DETAIL ①
DETAIL ②
DETAIL ③
DETAIL ④
Walk-way
El. 83.85 m.
B_E
A_E
A_W
B_W
C_W
D_W
Buttress Axes
6 Pipes to Well No. 10
6 Pipes to Well No. 5
40 30 20 10 0 10 20 30 40 50 60 80 100 120 140 160 180 200
Distance in Feet
PLAN
Center Line of Buttress A_E
3 Spaces @ 6″
3′-9″
DETAIL 1
Center Line of Buttress A_W
4 Spaces @ 6″
DETAIL 2
4″ Double Galvanized and Dipped Pipe
Riser
1:2 Mortar
Sand
5/16″ and 1¾″ Gravel
Varies
DETAIL OF COLLECTING BASIN
6 Pipes carried to Well No. 5 at El. 59.75 ± m.
6 Pipes carried to Well No. 10 at El. 59.75 ± m.
5 Spaces @ 6″
80′ South of Axis
DETAIL 3
8 Spaces @ 6″
DETAIL 4

Fig. 5.—Plan Showing Location of Riser Pipes for Foundation Testing System, Rodriguez Dam.

The arch barrel resembles a frame type of structure, which was adopted as the most suitable because of the large concentrated buttress loads. The stresses were computed by the elastic theory. The procedure followed in designing the several theoretical sections of the arch barrel was, as follows: Having determined the thickness and other dimensions of the arch rib by trial computations at a given section of the arch barrel (on the basis of its span and rise), the pressure line for the loads and weights was drawn and the arch axis made to coincide with this pressure line. The direct compression stresses at the crown and at the abutment could then readily be computed. The stresses due to rib-shortening, shear, and temperature variations were then calculated by the method of least work. These stresses, added algebraically to the direct compression stresses, gave the maximum and minimum stresses at the crown and at the abutments. The half section of the arch was divided into ten hypothetical voussoirs, and the stresses were computed at each voussoir point.

The working loads adopted for the arch-barrel designs were, for maximum compression, 700 lb. per sq. in., and for maximum tension, 50 lb. per sq. in. The final designs showed no tension in any arch section and no reinforcement was used.

The intended minimum compressive strength of concrete at 28 days, adopted for the mat, for the arch barrel, and for the 5-ft. bracing walls, was 2 000 lb. per sq. in., and that for the sub-intradosal concrete and the haunch filling, 1 200 lb. per sq. in. At the beginning of construction, the volumes of materials used per batch, in making concrete of the intended strength of 2 000 lb. per sq. in. at 28 days, were as follows:

Cement	8 sacks
Sand	18.3 cu. ft.
Gravel:	
$\frac{5}{16}$ to $1\frac{3}{4}$-in.	13.8 cu. ft.
$1\frac{3}{4}$ to $3\frac{1}{2}$-in.	13.8 cu. ft., or 24.0 cu. ft.
$3\frac{1}{2}$ to 6-in.	13.8 cu. ft.
Water-cement ratio	1.0

Afterward, it became evident that the desired strength could be obtained with a greater water-cement ratio, and the ratio was increased to 1.08. The use of the greater ratio made it possible to reduce the quantity of cement per batch to 7 sacks without reducing the workability appreciably. In the concrete for the sub-intradosal blocks, the volumes of materials per batch first used were: 6 sacks of cement and a water-cement ratio of 1.25. The sand and gravel quantities were the same as for the 2 000-lb. concrete. The number of sacks of cement per batch was reduced afterward to $5\frac{1}{2}$, with no change in the water-cement ratio.

The design of the transition structure between the flexible arch barrel and the rigid cut-off wall was a problem of some difficulty. The transition is shown in cross-section by Fig. 4. It consists of a massive block of concrete having a vertical depth ranging from 40 to 50 ft. and spanning the entire stream bed. The block overlaps the cut-off wall and is tied by steel reinforcement to the arch barrel. The bottom is reinforced longitudinally with three layers of $1\frac{1}{2}$-in. square steel bars, spaced 12 in., center to center, to resist possible tension due to bending in areas where the natural foundation has insufficient supporting power. It is reinforced at its easterly end by seven courses of $1\frac{1}{8}$-in. square diagonal tension bars, spaced 24 in., center to center, to aid in carrying the load at the fault to the solid rock on the eastern bank. It is also reinforced for possible diagonal tension due to its support by the cut-off wall.

CUT-OFF WALL

When the fault in the stream bed was discovered, it was realized that it would be necessary to carry the cut-off wall at the fault to a great depth. There was considerable doubt, however, regarding the depth to which it would be necessary to construct the wall in the western portion of the stream bed.

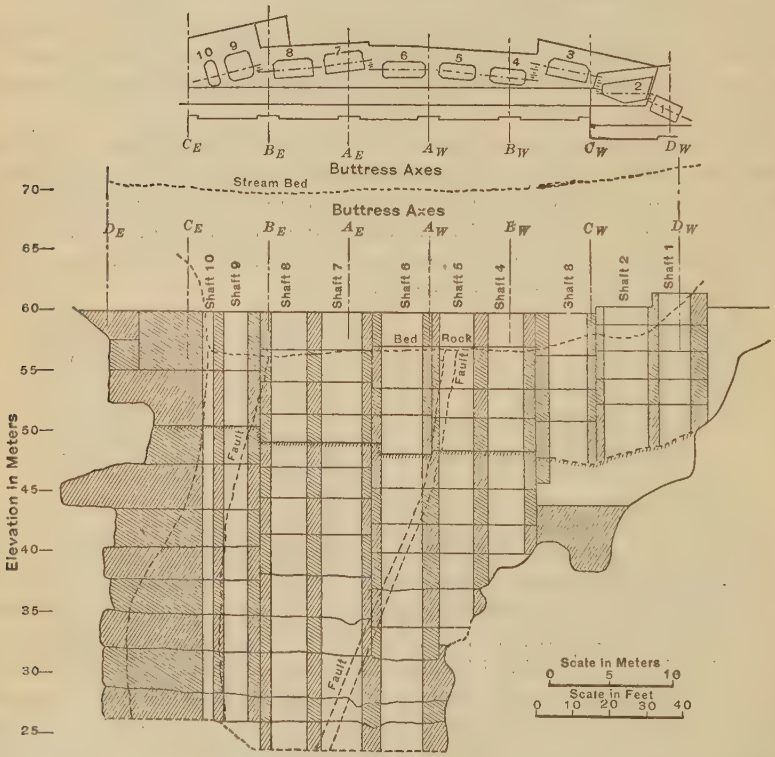
10
9
8
7
6
5
4
3
2
1
C_E
B_E
A_E
A_W
B_W
C_W
D_W
Buttress Axes
Stream Bed
Buttress Axes
D_E
C_E
B_E
A_E
A_W
B_W
C_W
D_W
Shaft 10
Shaft 9
Shaft 8
Shaft 7
Shaft 6
Shaft 5
Shaft 4
Shaft 3
Shaft 2
Shaft 1
Bed
Rock
Fault
Fault
Fault
Elevation in Meters
70
65
60
55
50
45
40
35
30
25
Scale in Meters
0
5
10
Scale in Feet
0
10
20
30
40

FIG. 6.—CUT-OFF WALL, PLAN AND LONGITUDINAL SECTION AT RIVER CHANNEL, RODRIGUEZ DAM.

Excavation of the cut-off trench was begun in open cut and was carried by this method to a depth of from 26 to 33 ft. below the original rock surface, corresponding to depths of from 66 to 72 ft. below the stream bed. The greater part of the trench required timbering. When the depths mentioned had been reached, maintaining the open cut had become so difficult that a change of method was necessary, and it was decided to continue the excavation through shafts. The trench was filled with concrete, in which ten shafts were constructed, through which the excavation and the placing of concrete were continued. The shafts are shown in plan and section by Fig. 6. The walls of the shafts were reinforced throughout the zone of inferior rock and well into the solid rock on either side. Stop-water key-ways were constructed in the walls, and, when excavation shall have been completed, the shafts will be filled with concrete or puddle, as then may be considered desirable.

Numerous grout pipes were inserted in the shaft walls, where the rock was broken or seamy, and two lines of grout holes, staggered in two rows 5 ft. apart, and spaced not exceeding 10 ft. apart in each row, were drilled from the bottoms of the shafts throughout that part of the length of the wall, where final depth has been reached. It is planned to drill numerous grout holes adjacent to and eastward from the fault, along the line of the cut-off wall extended, where the rock, although hard, is more or less broken.

Provision was made for the installation of twelve pressure pipes for determining the hydraulic gradient at various points on the up-stream and down-stream faces of the cut-off wall.

The length of wall constructed through shafts was, at the beginning, about 150 ft. At a depth of about 66 ft. below the original rock surface, the length had decreased to about 110 ft. Excavation and concrete shaft-walls have been carried to a depth of about 112 ft. below the original rock surface, or 150 ft. below stream bed, at which depth the length is about 90 ft. It is intended to carry the wall at the fault, and as far on either side as may be found necessary to secure a key in suitable rock, to a depth of 300 ft. below stream bed.

DISCUSSION

L. A. Robb,[8] M. Am. Soc. C. E. (by letter).—The foundation arch, the side-hill bracing walls, and the deep cellular-wedge type of cut-off wall of the structure described by Mr. Williams in his timely and interesting paper are innovations in dam design that may have a far-reaching effect. Faulted foundations and broken-rock banks, such as have been discussed by the author, will be encountered more frequently in the future because sites consisting entirely of sound rock are becoming increasingly scarce; and yet the development of these faulted and otherwise unsatisfactory sites are frequently of great economic importance.

Certain principles of foundation design and construction as presented by the author, with modifications, may be applied broadly to other difficult foundation problems of a similar nature. Aside from foundations which uniformly have low bearing value, such as clay or sand, difficulties are more frequently being encountered with: First, those foundations in which there exist the danger of horizontal or inclined movement, owing to seams and planes of cleavage; and, second, foundations consisting of materials varying widely in bearing value.

In the first type, movement may occur from earth or rock slides in the bank, due to planes of cleavage in the foundation material, or from more or

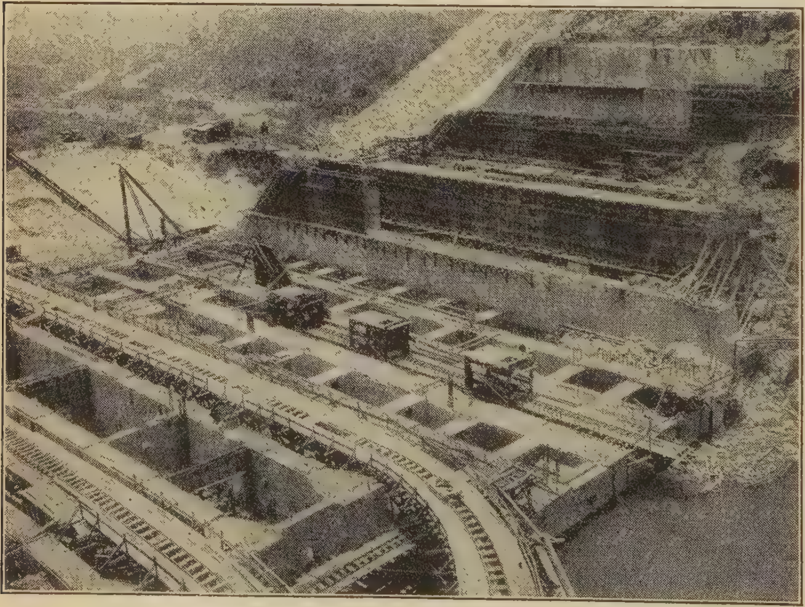

Fig. 7.—View Southwest from No. 2 Concrete Plant.

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[8] Vice-Pres., Ambursen Construction Co., New York, N. Y.

less horizontal planes of cleavage in the rock below the dam foundations. Such movements caused the failures of large gravity dams at both Austin, Pa., and Austin, Tex.[4]

Foundations of this type require that the seamed rock banks be braced either across the valley or anchored firmly to sound rock below in such a manner that the seams are water-tight and that lateral or longitudinal movements will be impossible. At the Rodriguez Dam the method of bracing the seamed rocks across the valley was used, as shown by Fig. 7.

Similar shattered bank conditions were encountered at another site recently studied. However, in this case, the valley was too wide to use bracing ribs. In addition, this problem was complicated by the fact that the side hills were of such height that loosened or broken material would have made excavation extremely hazardous. A foundation structure, involving a modification of the cellular-wedge cut-off construction described by the author, was worked out as a possible solution for this problem.

Fig. 8 shows the suggested design. It was proposed to construct four lines of the cellular-wedge type of cut-off walls, starting in the river bed and working upwardly into the bank. It was also proposed to carry the foundations of these walls well into sound rock and to converge the four lines as shown by Fig. 8 so that the up-stream and down-stream walls would coincide with the heel and toe of the dam. The footings are designed as an integral part of these walls, creating in effect a large subterranean reinforced concrete frame which would be capable at conservative stresses of carrying any possible thrust from the shattered or seamed side hill to the sound rock in the river bed below, and, in addition, would firmly anchor the dam to a sound foundation throughout its entire structure.

A foundation structure of this type could be constructed safely because a dangerous slide would be prevented during construction by the bracing action of the struts. No shoring or expensive bracing is required, and only a small amount of unsupported excavation is exposed at one time.

Several modifications of the arch-barrel design described by the author may be used in a treatment of foundations having widely different bearing values. It is necessary in such cases to span the areas of poor foundation with structural members that are supported by sound rock on each side, such as was the case for the Rodriguez Dam. An alternate type of structure, such as Fig. 9, might be used to advantage in other projects.

This type consists of an arch barrel or frame supporting the buttresses. Directly below the buttresses are heavy piers which, in turn, rest on a massive concrete mat spreading over the poor foundation material. This mat acts as a chord member to the arch and piers creating in effect a Vierendeel truss. The arch barrel is capable of taking all loads from the dam and the reservoir above. However, under normal conditions, the poor foundation material will actually take a substantial part of the load. The merits of such a structure are substantially the same as those of the type described by Mr. Williams.

The type of cut-off wall adopted for the Rodriguez Dam was also used in connection with the construction of a 30-ft addition of height to the Bristol

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[4] *Engineering News-Record*, October 19, 1911.

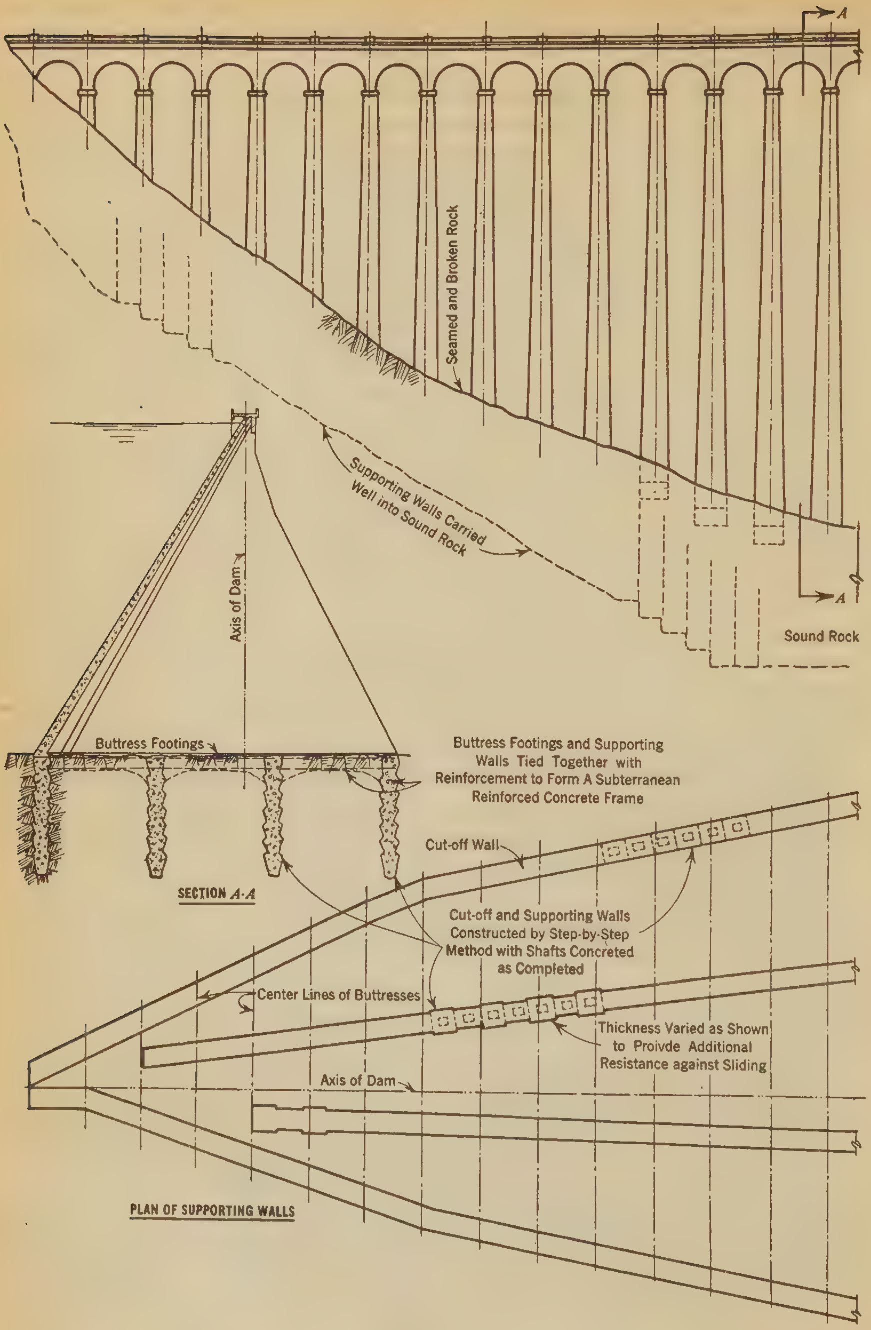
A
Seamed and Broken Rock
Supporting Walls Carried Well into Sound Rock
A
Sound Rock
Axis of Dam
Buttress Footings
Buttress Footings and Supporting Walls Tied Together with Reinforcement to Form A Subterranean Reinforced Concrete Frame
SECTION A-A
Cut-off Wall
Cut-off and Supporting Walls Constructed by Step-by-Step Method with Shafts Concreted as Completed
Center Lines of Buttresses
Thickness Varied as Shown to Proivde Additional Resistance against Sliding
Axis of Dam
PLAN OF SUPPORTING WALLS

Fig. 8.

Dam on the Pemigewasset River, near Bristol, N. H. At the east bank of this dam ledge rock slopes away from the ground line instead of following the more usual course of sloping in the general direction of the earth contours above. In order to provide adequate resistance against percolation through this bank a cut-off wall was required which increased in depth as the wall is carried into the bank, until the foundation of the wall was in ledge rock about 110 ft below the ground surface.

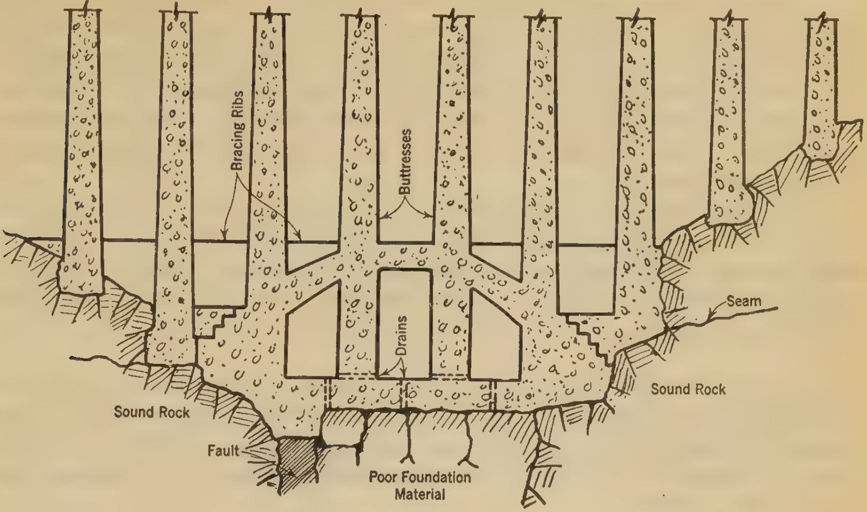

FIG. 9.

The concrete cut-off for the Bristol Dam was carried down through clay, gravel, and boulders, at the rate of about 1 ft in depth per day. Provisions were made to grout the joints between the sections of wall. The grout pipes also served as drains for the purpose of relieving hydrostatic pressure during construction. The advantages of using this type of wall are, as follows:

1.—The necessity of excavating a wide, deep trench and of using expensive and dangerous shoring is avoided.

2.—Wedge action firmly compacts the material supporting the wall and thus greatly increases the resistance of the wall to water travel.

3.—This procedure offers a safe and rapid method of constructing a deep cut-off wall and involves a minimum amount of exposed excavation.

4.—The wall is not endangered by flood hazards while it is in process of construction. If in the river bed, during the flood season, the cellular spaces may easily be protected against filling with silt and débris and only a minimum amount of pumping will be required to empty the cells after the flood has passed, or the shafts may be carried up above flood-water level and construction continued uninterruptedly.

FRANCISCO GOMEZ-PEREZ,[5] JUN. AM. SOC. C. E. (by letter).—In his paper the author has covered his subject in so thorough and complete a manner that it is difficult to add anything without having to enter into other phases of the construction which are independent of the foundation and, therefore, not within the scope of the discussion.

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[5] With Mexican National Irrig. Comm., City of Mexico, Mexico.

A point of view implied in the paper but which, because it is not of a technical nature, is not developed, brings attention to the fact that even in the face of difficulties of various kinds, Mexico is doing what she can to proceed with the construction of public works. These include some of National scope, such as irrigation developments, in which members of the Society have co-operated. The technical aspects of some of these projects have been published by the Society.

It is to be hoped that the association of engineers from the United States, endowed with such a high technical and cultural background, and such personal integrity, with Mexican engineers, will tend toward mutual understanding and appreciation, which, in time, will bring about a more cordial and effective co-operation.

F. A. Noetzli,[6] M. Am. Soc. C. E. (by letter).—The unusual foundation conditions at Rodriguez Dam, which involve a geologic fault about 20 ft wide, presented a problem that could be met only by unusual methods. Mr. Williams has presented a clear and concise description of how the difficulties were successfully overcome.

The main feature of the foundation structure is a horizontal foundation arch that bridges the bottom of the canyon with a clear span of, roughly, 70 ft and extends for a distance of approximately 250 ft in an up-stream and down-stream direction. This long arch barrel supports four of the highest buttresses of the dam.

At first, the problem of the design of a foundation arch for supporting buttresses nearly 250 ft long, seemed to present almost insurmountable difficulties from a theoretical point of view, mainly because of the necessity of taking into account the arch deflections resulting from the heavy concentrated loads exerted by the buttresses.

The foundation arch had to meet the following requirements: (*a*) The axis of the arch barrel was to be parallel to the buttresses so as to avoid undesirable twisting moments; (*b*) the span, radius, and thickness of the elementary arches of the arch barrel were to be of such a size that the theoretical arch deflections along the base of each buttress would be as nearly as practicable on a straight line and would correspond to the trapezoidal stress at the base of the buttresses.

Fortunately, the bed-rock conditions along both side hills at the bottom of the canyon made it possible to meet these requirements very closely. For some parts of the arch barrel it was necessary to use arch spans of greater length than would have been necessary to span the fault and the rock of inferior quality, and, for certain other parts, some extra excavation was required; but, in general, the conditions may be said to have been rather favorable for a foundation structure of the type adopted.

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[6] Cons. Hydr. Engr., Los Angeles, Calif. Mr. Noetzli died on May 24, 1933.

The structural analysis of the individual elementary arches was made by the elastic theory and offered no unusual difficulties. For the purpose of design the arch barrel was divided into strips from 10 to 20 ft long. The radius, span, and central angle, and the thickness of each strip at crown and abutment were then determined by "cut and trial" such that the necessary conditions regarding deflections and permissible maximum unit stresses were met within reasonably close limits. It is of importance to note that the buttress walls that transmit the load to the foundation arch are relatively stiff in a vertical direction. They will tend, therefore, to force a practically linear deflection of the foundation barrel to occur along the base of each buttress, regardless of the results of the theoretical analysis. For small differences between theoretical and actual straight-line deflections the loads on the individual strips will depend upon their relative stiffness, such that the stiffer parts will carry somewhat more than the theoretical load, but will thereby relieve the relatively more flexible parts by the same amount of load. For instance, if one strip of the arch barrel were overloaded by, say, 5% of its theoretical load, the adjoining strip or strips would be relieved by approximately this same amount so that under actual load conditions the deflections of the arch barrel would again vary practically linearly along the base of each buttress. For safety, the lower part of each buttress is strongly reinforced, as shown in the author's Fig. 3.

While the foundation arch is designed to carry the full superimposed load, no doubt a considerable part of this load will be supported through the subintradosal concrete by the natural foundation material in the bottom of the canyon.

Small irregularities in settlements of one buttress relative to the others are not likely to affect the stability of the dam, because of the relative flexibility of this type of structure.

One interesting feature of the foundation structure of the Rodriguez Dam is the provision for admitting water through inlet pipes along the intrados of the foundation arch, whereby artificial uplift pressure is created which, in periods of floods, counteracts the heavy load on the foundation arch, due to back-water entering between the buttresses.

Charles P. Williams,[7] M. Am. Soc. C. E. (by letter).—In view of the somewhat radical departure from conventional types of foundations for buttressed dams, the writer would have appreciated a more critical discussion by those who heard, or have read, the paper and have viewed the work during construction. Nevertheless, he appreciates the comments of those who entered into the discussion.

The building of any storage dam is a serious responsibility, especially in a region subject to earthquakes and at a site traversed by a geologic fault. The construction at such a site necessarily involves some risk. However, the region around the Rodriguez Dam is dependent absolutely upon storage for its water supply. Ground storage sufficient to meet the requirement cannot be developed. Periods of five years in which there is no stream flow are of common occurrence. Records show one period of seven years during which the run-off was insignificant. The construction of reservoirs at the best, or at the least objectionable, sites is a necessity. The question is whether the risk is too great to justify continued development and, if not, what type of structure shall be built in order to secure the greatest practicable insurance against damage.

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[7] Cons. Engr., Los Angeles, Calif.

No comment on the discussions submitted appears to be necessary. At the time of the presentation of the paper at the meeting of the Irrigation Division, in Sacramento, Calif., in April, 1930, some comment was presented relative to the geology of the site. It was suggested that the rift in the bedrock at the Rodriguez site is not a true fault, but a tensile crack, the main fault lying between the mainland and the Coronado Islands; that the type of dam selected is not suited to the site; and that a more flexible type, such as an earth or rock-fill dam, should have been chosen.

The fact that the rift at the dam site is a true fault, and not a mere tensile crack, is proved by the occurrence of dense clay gouge, as much as 1 ft thick, along the faces of the hanging wall and foot wall.

At the time of the preliminary examination, the writer suggested an earth and rock-fill dam as being the type probably best suited to the site. This recommendation was made without knowledge of the existence of a fault, but with the knowledge that borings indicated, for a part of the foundation area in the stream bed, underlying disintegrated material extending to a depth of more than 100 ft. The writer had no part in the selection of the type of dam which eventually was chosen; however, in the light of subsequent determination of the character of the foundation site, he is of the opinion that the selection was fortunate. Excavation disclosed the fault, not discovered in the preceding geological examination. The question of shearing movement along the fault then became one of paramount consideration.

The two geologists who examined the site, after the discovery of the fault, agreed that subsequent movement along the fault is not probable, although it is a possibility. If, then, a dam is to be built at such a site, the question arises as to the type of structure which will restrict the damage to the least possible amount should geologic movement occur. Such a movement might be either a vertical or a horizontal slip, or it might be a separation of the fault walls, forming a crack.

The effect of the earthquake of 1906 on the San Andreas and Upper Crystal Springs (earth) Dams, near San Francisco, Calif., is sometimes cited as evidence of the relative safety of earth dams from damage by seismic disturbance.[8] At the time of the 1906 earthquake, the San Andreas Dam, which then had a maximum height of 95 ft above the stream bed, consisted actually of two dams, separated by a hard narrow ridge through which passes the San Andreas fault. A horizontal slip of about 8 ft occurred at the dam, but this slip was along the fault and did not pass through either of the dams. The structures were somewhat cracked by the earthquake, but failure did not occur although the reservoir was full at the time. The results would have been very different, however, had the slip passed through the dam proper. The Upper Crystal Springs Dam, having a maximum height of 85 ft above the stream bed, is crossed by the San Andreas fault. At the time of the 1906 earthquake, there was a horizontal slip of 8 ft through the body of the dam. This dam lies between the Upper and the Lower Crystal Springs Reservoirs and, at the time of the earthquake, the water stood at the same elevation above and below the dam. Very different results might have been expected, if the dam had been subject to unbalanced hydrostatic pressure.

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[8] *Engineering News-Record,* August 25, 1932, p. 218, and September 29, 1932, p. 385.

It is inconceivable that a slip, such as occurred at the San Andreas and Upper Crystal Springs Dams, could occur in an earth dam of the height of the Rodriguez Dam, with a large quantity of water in storage, without failure resulting from the admission of water into the body of the dam through cracks, or, possibly, in the case of a vertical slip, by overtopping. After initial failure, progressive failure would be rapid, resulting in early collapse. The objection to an earth dam, at this site, applies, in a somewhat less degree, to a rock-fill dam.

The Ambursen type of dam is more flexible than any other type of concrete dam, and small relative movements can occur without resulting in serious torsional stresses. If, however, a slip along the fault should occur, there is possibility of local failure. The failure of one or two buttresses might occur, without affecting seriously the remainder of the structure, except by progressive undermining, which probably would be slow. The dam might be even torn apart, but the separated sections, not being dependent for stability on the part destroyed, would still stand and would retard to a considerable extent the escaping water.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1864

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

By Robert Fletcher,[1] and J. P. Snow,[2] Members Am. Soc. C. E.

With Discussion by Messrs. Henry B. Seaman, Jasper O. Draffin, E. K. Morse, John W. Storrs, C. J. Hogue, William A. Oliver, Philip G. Laurson, Richard S. Kirby, Wells N. Thompson, Benjamin Wilder Guppy, Ivan E. Houk, William H. Baker, Jr., Oren Reed, R. P. Davis and L. L. Jemison, Peter T. Landsem, J. K. Finch, A. A. Eremin, Aksel Anderson, William G. Atwood, H. Fugl Meyer, W. E. Howland, Harry K. Ellis, Nelson J. Bell, and Robert Fletcher and J. P. Snow.

Synopsis

This story of wooden bridges is written from the engineer's viewpoint. Other writers have presented the æsthetic, non-technical, and sentimental aspect of the subject. Beginning with the dawn of history, the story ends with the evolution of types of timber bridges accomplished only within the two centuries just past; and this development has culminated in the United States, giving to this country the latest types designed in conformity with correct principles and built at minimum cost. In the treatment of the subject three periods may be recognized.

The first period covers what it is now possible to learn concerning wooden bridges constructed by so-called "ancient" people and includes the indefinite period indicated by the historian's phrase, "the fall of the Roman Empire." There seems to be reason for the opinion that types of primitive bridges have survived many centuries in the "unchanging East," and only recently have come distinctly to the knowledge of the Western civilized world. Descriptions of such types are given.

Note.—Published in November, 1932, *Proceedings.*

[1] Director Emeritus, Thayer School of Civ. Eng., Dartmouth Coll., Hanover, N. H.

[2] Cons. Engr., Boston, Mass. Mr. Snow died September 4, 1933.

The second period covers the development in Europe from the so-called "Dark Ages" to about the Sixteenth Century, when scientific knowledge began to influence the minds of mankind—the period of the Renaissance. Then, beginning about the middle of the Eighteenth Century, a variety of types appeared, showing much ingenuity in design and skill in construction. It was a time when both rulers and people gave more attention to public works of all descriptions.

Finally, the modern period deals at greater length with the various stages of development through experimental types of American timber bridges to types now accepted as standard. Such are principally those which make use of lattice, Howe, and Pratt forms of truss, with or without a timber arch in combination.

Presumably the end of the period would be when steel and concrete have displaced timber in bridge construction; but it will appear that, where timber is plentiful and considerations of economy and rapid accomplishment prevail, wooden bridges still have "a reason to be." A review of the dominant facts established is given in the "Summary."

INTRODUCTION

Extensive inquiries made in twenty-four (only one-half) of the United States (Eastern, Middle, Southern, and Central) have revealed the surprising fact that, as late as 1930, there were between 450 and 500 covered bridges in use, a goodly number of which are likely to serve for some decades to come. A quarto volume, published in 1931, gives 135 illustrations of such bridges.[3] The ultimate settlement of a large area of undeveloped country, such as Alaska, South America, and parts of the Eastern Hemisphere, where timber is abundant, will make demands for bridges that can be built with timber, quickly and cheaply, and that will be durable enough for the pioneer period.

Originally, much of the area of the United States was heavily forested, especially the Eastern part. Naturally the first settlers used the timber at hand for bridges and other construction. The shipyards along the New England Coast, where so many fine ships were built, were effective training schools for heavy timber framing; and the influence of that training still remains among the carpenters of the region. Those conditions led to the building of many bridges of note during the period of development and trade after the Revolutionary War. The spirit of emulation and invention which is characteristic of the American people resulted in a flood of patents for bridge-truss types, a few of which have led to good designs which have been widely approved and adopted. The full text of the investigation upon which this paper was based has been placed on file with Engineering Societies Library, in New York, N. Y., to serve as a possible aid to any subsequent research on this point.

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[3] "Covered Bridges in America," by Rosalie Wells, with foreword by Charles S. Whitney, M. Am. Soc. C. E., New York, William Edwin Rudge, 1931.

The Ancient Background

Undoubtedly the prehistoric man built his bridges with the adaptable materials within his environment. Where trees abounded, a log fallen or felled so as to bridge a stream, would be a suggestion to the most primitive intelligence. Hence, the simple beam bridge, under favoring conditions, would be a first resort and, from prehistoric time to the present, may be regarded as a world-wide type.

Again, if the tale[4] of an early explorer in South America may be believed, even the monkey may have taught the prehistoric man. In a tropical forest he may have seen a chain of monkeys, their bodies locked together by means of their long arms and long tails, hanging from the branch of a tall tree and swaying until the lowermost grabbed a branch on the other side of a stream. Then the crossing of the group on this living bridge, the shifting of the point of attachment on that side, the reverse swing of the chain until the rearmost monkey landed, and, finally, the breaking of the chain would be quite an effective demonstration, if and when witnessed.

In the sub-tropical parts of Central Asia, claimed to be the cradle of the human race, bountiful Nature provided ample material for constructing suspension bridges. There is the rattan, or climbing palm, a species of the *genus calamus,* remarkable for the length of its stems growing out from a slender trunk.[5] These stems are very tough, having hooks by which they climb over bushes and trees; sometimes they attain a length of 300 to 800 ft., or more.

Another adaptable material is found in the liana, defined as any perennial plant having woody stems, which are characteristic of certain geographical regions, as the forests of the Amazon, the jungles of India, etc.[6] In the temperate zone the grapevine is an analogous example. The grass rope is a species of bamboo that belongs to the great grass family.

Rope bridges may be classified into five types: (1) Tight rope; (2) single rope and dragline; (3) toboggan; (4) roadway; and (5) tubular type. The tight-rope type, consists of a single rope to serve as a footwalk and two side ropes for hand-rails. This type has been mentioned by the English traveler, E. Kingdon-Ward, to illustrate his article[7] on "Bridge Builders of the Jungle." It is a remarkable fact that this same type of bridge was found by an American explorer in the Cameroon—the French mandate in Equatorial West Africa.[8] It was made safer, however, by a woven net of vines extending from the foot-rope to both hand-ropes. This was among the negro race, so far distant from the Aryan people of the high plateau of Central Asia that it is difficult to understand how, even in the long course of history, there could have been any contact with or communication between them.

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[4] Described in *Merry's Museum,* a serial magazine for young people, more than sixty years ago. Recollection verified by a contemporary reader. Encyclopedia Britannica, Fourteenth Edition, Vol. 18, Article "Primates," pictures the "spider monkey of tropical America, with long arms and long prehensile tails." These would be capable of the performance described.

[5] Appleton's Cyclopedia, Vol. XIII, Article, "Palm."

[6] Webster's International Dictionary.

[7] *Travel,* February, 1931, p. 38.

[8] *National Geographic Magazine,* Vol. 59, February, 1931, p. 231.

In Tibet, rivers are crossed by a single rope which serves, by aid of a trolley block and sling, and a dragline, to transport singly a man, an animal, or a parcel across a stream. (Type (2).)

The toboggan type of bridge (Type (3)) was also described by Mr. Kingdon-Ward.[7] This is a single rope securely anchored to a post or tree higher up than the point of landing on the opposite side of the stream. On this, slides a trolley block from which is hung a sling to support the weight of the traveler. The rope and the block are lubricated by yak butter—a product of Asiatic cattle. Standing in the sling, with his hands grasping only the block, the traveler is started by a push, and the force of gravity rushes him across the torrent with fearsome rapidity. Of course, another rope is set for crossing in the opposite direction.

The roadway type (Type (4)), shown in Fig. 1, needs little description. This example is in the Island of Java—Dutch Colonial territory. Apparently, the span is about 100 ft., and the construction is quite like that which an engineer of to-day would make it with the material at hand.

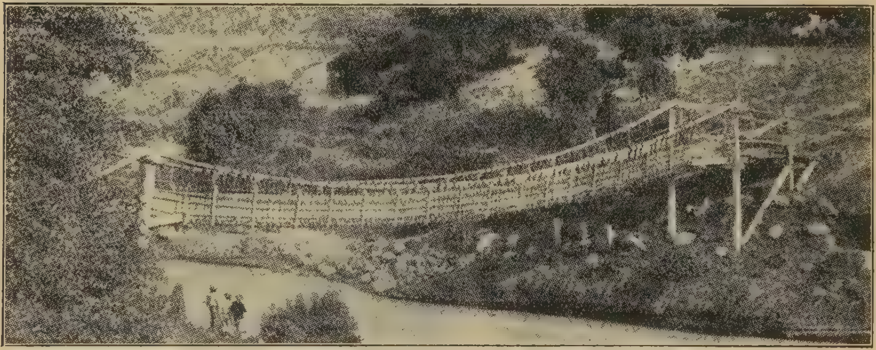

FIG. 1.—HIGHWAY TYPE OF ROPE BRIDGE, JAVA.

The tubular type (Type (5)), shown in Fig. 2, was another found by Mr. Kingdon-Ward among the fierce Abor people on the Assam frontier. It spans the Dihang River, and is nearly 800 ft. long, the center being 50 ft. above the stream. Report was that 200 men built it in about a month, the labor being freely supplied by the villagers who used it.

A variation of the roadway type of bridge is described by Spanish historians as found by the invaders of the Sixteenth Century in the Peruvian Andes.[9] These bridges were composed of four ropes of twisted vines, covered with branches for a floor, and two ropes for hand-rails. To cross comparatively still water several hundred feet wide, the Inca workmen laid two great ropes of twisted grass on the water surface, on which cross-fascines of dry rushes were lashed. Two other ropes were laid on the fascines, which were again covered with smaller fascines, making a floating bridge with a floor, 15 ft. wide, over which armies passed. Such a structure required renewal or repairs every few months. The Incas also used a single rope on which an under-slung hammock was held by clews so as to be drawn back and forth by a dragline. The widespread use of such "prehistoric" bridges in both hemispheres is significant of antiquity.

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[9] Edinburgh Cyclopedia, 1832, Vol. 4, p. 471.

A French historian maintains that types of bridges found among backward tribes or communities in the hinterland of more advanced civilization are survivals of prehistoric structures with little if any change.[10] In the hills of Savoy in Italy, a primitive bridge was found, the structure of which had changed very little since the time of Julius Cæsar, who described[11] similar structures used by the Gauls, thus:

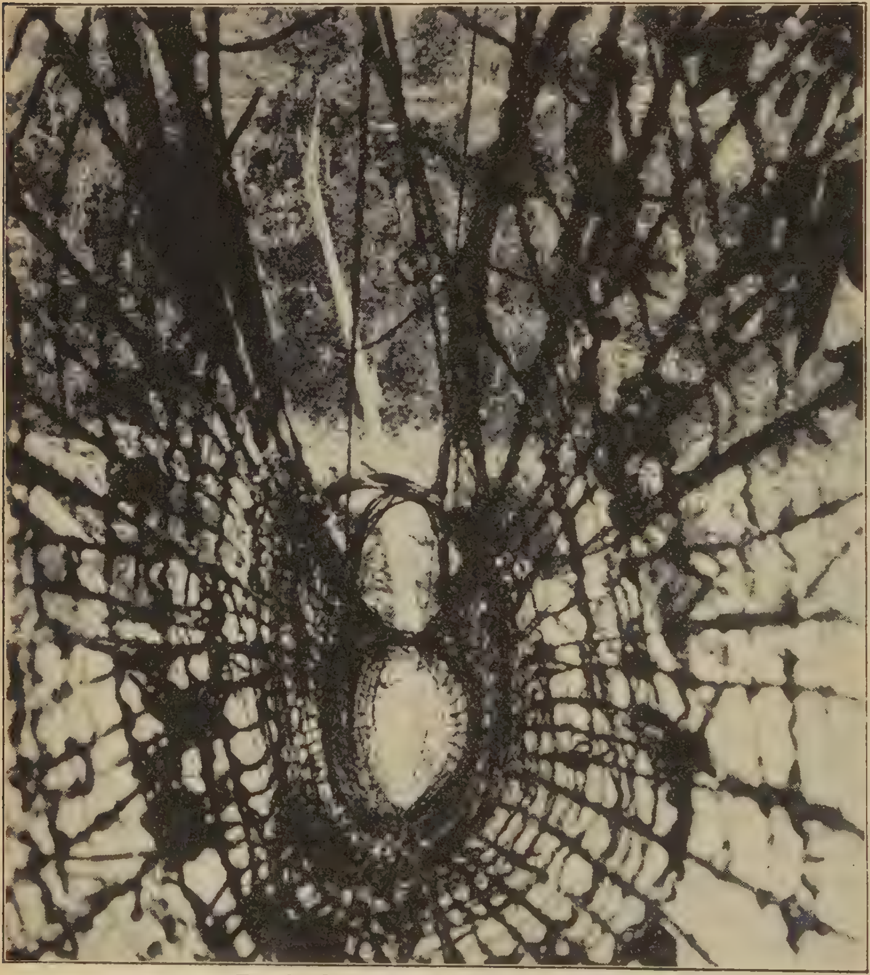

FIG. 2.—TUBULAR CANE BRIDGE, ASSAM.

"It is a timber bridge or empilage, piled together rudely, not constructed by art. It needs no carpentry. * * * On each bank of the stream a rough foundation of water-worn boulders was laid, about fifteen feet square; upon this a criss-cross (crib-work) of the tree trunks was built so that the logs in the direction of travel, in alternate layers, were made to jut out farther and farther over the water, narrowing the gap to be bridged later by a few logs serving as beams."

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[10] "A Book of Bridges," by Brangwen and Sparrow, Plymouth, England, 1914 (in Dartmouth College Library).

[11] So stated by the author quoted.

Then, a floor of smaller round cuttings laid cross-wise over all made, in modern phrase, a corduroy road. Examples of such bridges in Kashmir, Northern India, and in Bhutan Province, north of Bengal, India, have been published by Charles S. Whitney, M. Am. Soc. C. E. One of these is apparently a development of what may be termed the corbel type into a real cantilever type of bridge. The portal towers, which probably serve partly as anchorages, suggest similar portals of Mediæval Europe. The total span appears to be between 80 and 100 ft.[12]

If prehistoric types survive to-day they would be among people like those of the "unchanging East," little affected by time or political disturbance, where the repairs made necessary by the vicissitudes of climate, decay, floods, and destruction by fire or enemies could be quickly and continuously effected.

The early historical period has little to tell about wooden bridges for many centuries. A writer states[13] that the Greeks, although masters of architecture, built few bridges and points out that "even Athens, with its splendid edifices, had no bridge over the Cephissus which ran athwart the most frequented thoroughfare to the city."

As to the Romans, Messrs. Brangwen and Sparrow state[10] that it has not been possible to learn how the Romans built their timber bridges. Pons Sublicius was a sacred monument, hallowed by traditions; but even this bridge was imperfectly described. According to the aforementioned writers, the most that can be said is that the Pons Sublicius was a tree bridge resting on piles. It dated from the time of Ancus Marcius who reigned from 640–616 B. C. Whether the chief priests did or did not build it, they were faithful in keeping it repaired, since it was in existence as late as the reign of Constantine (306 to 337 A. D.)—at least 900 years.

However, a reliable description is available of the timber bridge built (probably) under direction of the distinguished architect and engineer, Vitruvius, about 2 000 years ago, which permitted the passage of the Roman Army into Germany. This is frequently described as Cæsar's bridge. In his day, Palladio, the Venetian architect (1518-1580), made an exhaustive study of the vestiges of the Roman empire.[14] In his treatise on "Architecture," he gives, in the Latin, Cæsar's description of this bridge and a drawing to show the method of construction. The simplicity of the construction is obvious from Fig. 3. Its ingenuity appears by a brief study of the details, which shows how the parts of the bents lock themselves by action of the notched block in the notches on the cross-girder and inclined struts. Imposed weight simply makes the joints tighter. Thus, these bents could be framed quickly and set in place and could be removed quickly without the use of more elaborate fastenings, which would require time and supplies of hardware. It is on record that the Emperor made Vitruvius Superintendent of *Ballistæ* (Chief of Artillery), and this temporary structure in its design gives evidence of such skill as Vitruvius would doubtless possess. It is scarcely probable that the lumber was as smooth and square as the drawing indicates.[15]

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[12] "Whitney's Bridges," by Charles S. Whitney, M. Am. Soc. C. E., Pl. 31, p. 55; Pl. 32, p. 56, Edwin Wm. Rudge, Pub., 1929.

[13] Appleton's Cyclopedia, Vol. III, p. 270, Article, "Bridge."

[14] "The Architecture of Palladio," Large folio, with many full-page engravings; translated from the Italian, Lond., 1721.

A little more than a century later (104 A. D.) Trajan's bridge across the Danube River is mentioned in Roman history as one of the most prodigious works ever undertaken by the Romans. Dion Cassius stated that it rested on twenty piers, 150 ft. high and 170 ft. apart. These were united by semicircular timber arches. Perhaps the height was fixed by the high banks of the river. Nicholas Bergier[16] states that the piers of squared-dimension stone were built upon rip-rap dumped upon the unstable bed of the stream and that the legionaries were generally put to work on the roads and bridges to avoid the evils of idleness. Hence, with the customary use of slaves and commandeered labor, the abundance of timber in a wild country, and transportation made easy by the river, work on several piers and timber arches could proceed simultaneously. The carving, showing a span of this bridge, on Trajan's Column in Rome, as illustrated in books, is probably only conventional, since it is not in accord with the description given. Bergier presents long inventories of the roads of the Roman Empire in its greatest extent, from Britain to North Africa, and from Spain to Asia Minor, showing a total of nearly 65 000 miles. As to wooden bridges he states that timber was used in the early stages of road extension, but such bridges were eventually displaced by structures of stone.

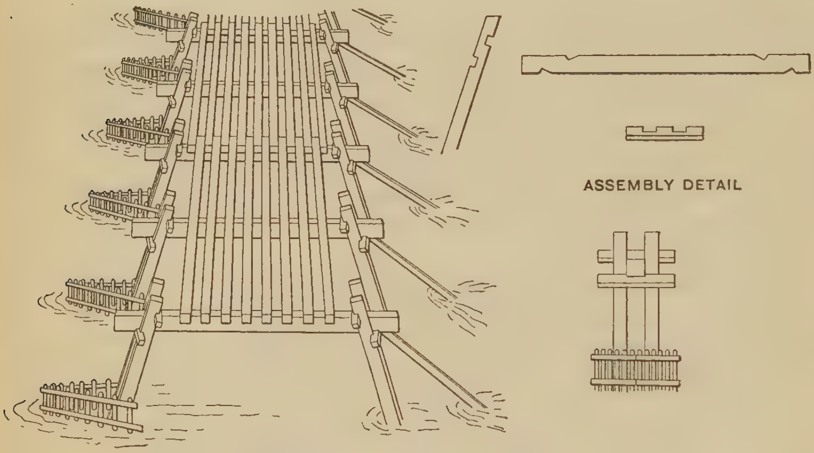

FIG. 3.—CÆSAR'S BRIDGE ACCORDING TO PALLADIO.

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[15] Appleton's Cyclopedia, Vol. XVI, p. 394; also, "Vitruvius," Encyclopedia Britannica, Fourteenth Edition, Vol. 23.

[16] "Histoire des Grandes Routes de l'Empire Romain," by Nicholas Bergier (2 vol.), Vol. 2, livre 4, Chapter 2, p. 610, pub. in Brussels, Belgium, in 1728. Nine chapters on Roman bridges.

LATER EUROPEAN TIMBER BRIDGES

The Sixteenth Century.—The first illustration of a framed truss, in the modern sense, that the writers have yet discovered, appears in Palladio's "Architecture" (Fig. 4). That author states that this truss spanned a mountain torrent in Northern Italy. Apparently, the span is between 50 and 60 ft. The vertical members he terms pillars, or *collonelli.* The peculiar connections for carrying the weight from the cross-girder to the posts are possibly of his "invention," because he describes three "inventions," one of which is similar to Fig. 4, but with curved chords; a second, which has little merit, and a third, the framed arch shown by Fig. 5.

His designs seem to have attracted little attention because there was no further development of timber bridges in Europe until the middle of the Eighteenth Century—a period of rapid progress, when both rulers and people gave more attention to public works of all descriptions and when Civil Engineering began to be recognized as a profession. In this development French engineers took precedence, publishing elaborate treatises on architecture, carpentry, civil constructions, etc. In France, Perronet built his magnificent monumental stone bridges which served as models for English engineers and for those of other lands. He was a Director of "L'Ecole des Ponts et Chaussées," in Paris. English engineers, lacking abundant timber,

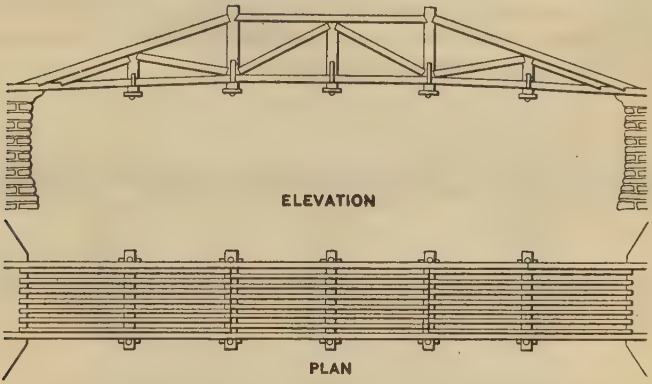


FIG. 4.—A TRUSS DATED ABOUT 1550 A. D.

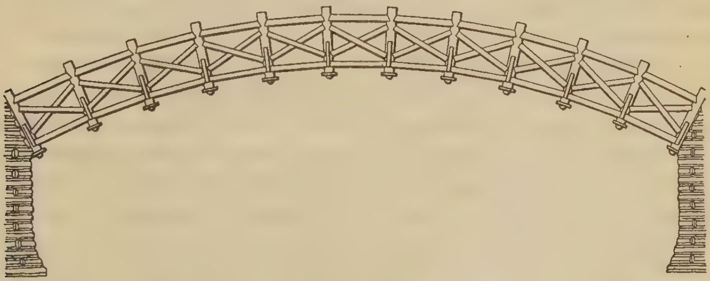
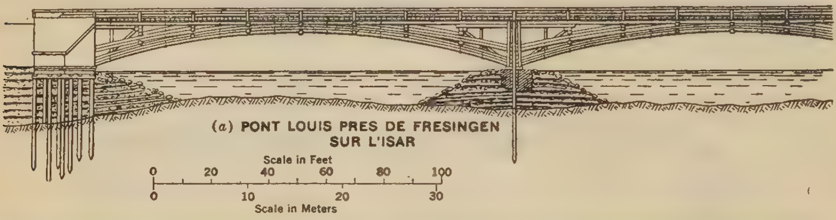


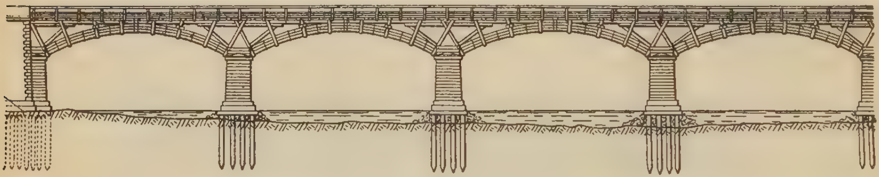
FIG. 5.—PALLADIO'S TIMBER ARCH.

utilized cast iron in some notable bridges standing to this day; but from the first they followed French practice in constructing some famous stone bridges.

The French types of timber bridges had the distinguishing characteristics shown in Fig. 6;[17] namely, level floors, low parapets, and flat arches, usually not framed after Palladio's pattern, but built of plank in layers clamped together. The floors were supported by radial struts attached to the arch and by braces from piers and abutments. The piers, usually of stone, were built at least to the skew-backs of the arches and sometimes to the floor level. The longer span shown (Fig. 6(*a*)) is about 150 ft.; the shorter one (Fig. 6(*b*)), is about 65 ft. In general, the designs produced pleasing architectural effects. Occasionally, piers were in the form of pile bents; and simple pile and beam trestles were resorted to when long spans were not needed.



(*a*) PONT LOUIS PRES DE FRESINGEN SUR L'ISAR



(*b*) PONT DE CHOISY SUR LA SEINE

FIG. 6.—EUROPEAN TIMBER BRIDGES, EIGHTEENTH CENTURY.

Covered or roofed-in bridges were distinctly not a feature in European practice. Two notable Swiss bridges, built by the Grubenmann Brothers, were exceptional. The picture of the Schaffhausen Bridge has appeared in so many books and atlases as to be familiar to engineers generally. Fig. 7[18] shows the greater part of one span, with the details fairly distinct. It has been overpraised. It was a ponderous combination of long and short braces extending from as many points as possible to both abutments—a maze of timbers, scarfed, bolted, strapped, and clamped together to form nondescript trusses. The roof was top-heavy, with a needless amount of timber. It had two spans of 171 and 193 ft., propped by abutment braces. It was finished in 1758 and destroyed by the French in 1799. The one at Wittingen, Germany,[19] had a single span of 390 ft., making a stretched-out and awkward truss lacking the appearance of adequate strength. Neither bridge was worthy of consideration from an engineering standpoint. The assertion of an English writer that these bridges suggested the idea of the American covered bridge, with its roadway between trusses, is not justified by the facts. The early American types developed in the last two decades of the Eighteenth Century, and later, have little resemblance to those structures.

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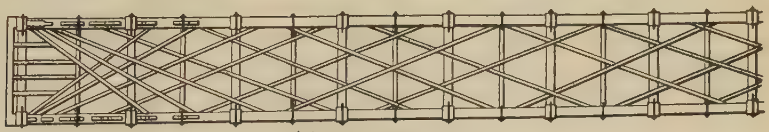
[17] From Gauthey's Atlas accompanying his 3-vol. treatise on "Constructions des Pont."

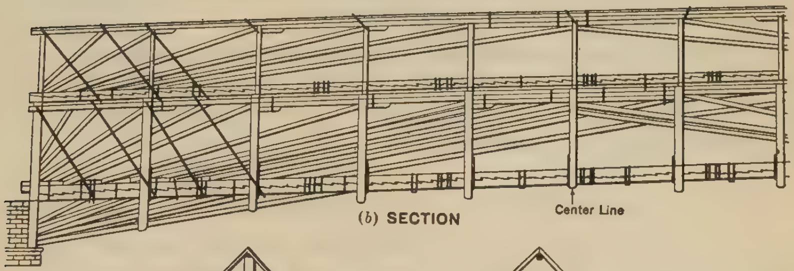
[18] "Notable Bridges"; Atlas of ten large folio engravings pub. between 1799 and 1825, by G. Taylor, 59 High Holborn, London, England.

[19] Gauthey's Atlas, previously mentioned.

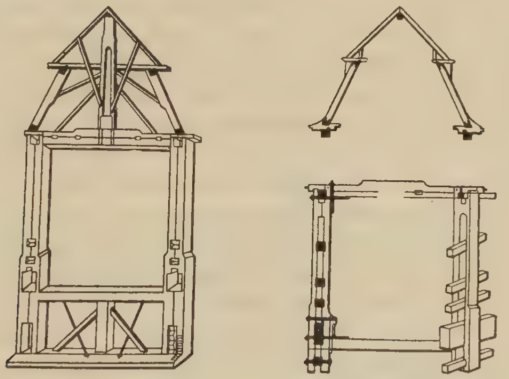
The only other European roofed bridges discovered[19] are: (1) The "Great Bridge" in Zurich, Switzerland, 128-ft. span, rather clumsily constructed with timbers laboriously scarfed from end to end and bolted together in pairs; (2) a bridge at Bassano, Italy, over the Brenta River, on pile bents in short spans, and covered by an open colonnade supporting a light roof; (3) a project by M. Gauthey for a span of 50 m., which is impractical and not worthy of consideration; and (4) and (5) two other bridges, such as local carpenters might devise. The drawings do not indicate side covering.



(*a*) PLAN OF FLOOR



(*b*) SECTION



(*c*) (*d*)

FIG. 7.—PLAN AND SECTIONS, THE SCHAFFHAUSEN BRIDGE, SWITZERLAND.

AMERICAN TIMBER BRIDGES

Development of Types.—The first settlers along the Eastern Coast of America found themselves in comparatively flat country, intersected by broad estuaries and tidal marshes. These conditions sometimes required bridges of considerable length and low elevation which were readily and inexpensively constructed by adopting pile bents and short spans. As loads were light, beams of moderate size were sufficient.

An authoritative history of this development was begun by the late Theodore Cooper, M. Am. Soc. C. E., in a paper presented to the Society more than forty years ago.[20] His short section on wooden bridges begins with the year 1660, thirty years after the settlement of Boston, Mass. He mentions "The Great Bridge" built in 1660, across the Charles River, which was 270 ft. long, on 13 piers. Twenty-five years later, another bridge was built over the same river, 1 503 ft. long, on 75 piers; the West Boston Bridge over the Charles River, 3 500 ft. long, on 180 pile bents, was finished in 1793; and in the same decade a pile bridge, a mile long, in 25-ft. spans, was erected over Cayuga Lake in New York State. In 1792, a bridge was built by Col. William P. Riddle over the Merrimac River, at Amoskeag Falls, now Manchester, N. H. This bridge was about 556 ft. long, supported by five cribwork piers filled with stone, and connected by beams not longer than 40 ft., which were then readily procurable from the great pine forests. Even before that, the Northwestern frontier had been pushed into Vermont, and a bridge was needed over the Connecticut River.

Colonel Enoch Hale's Bridge

In 1785, Enoch Hale built a notable bridge over the Connecticut River, at Bellows Falls, Vt. He was born in 1735 at Rowley, Mass., and moved to Rindge, N. H., in 1760, before it was incorporated as a town. He soon became prominent in the community, holding finally the office of High Sheriff of Cheshire County. In 1783, he obtained a grant from the Legislature to build a toll bridge over the Connecticut River between Walpole, N. H., and Rockingham, Vt. In 1784, he moved his family to Walpole and completed the bridge in 1785, two years after the close of the Revolutionary War. The bridge crossed the gorge at Bellows Falls, the narrowest point of the river where there is a rock reef dividing the water into two channels. This was the first bridge over the river at any point and while it was being built, the residents looked upon it as a foolhardy experiment. It is reported by contemporary newspapers to have been 365 ft. long, in two spans, supported at the center on the natural rock pier.

Fig. 8 is a sketch from an oil painting of the locality, showing the original structure, or a successor to it. The date of the painting is uncertain. Undoubtedly, the type shown represents that built by Enoch Hale. It consisted of four lines of braced stringers said to have had spans of 98 ft. each at the foot of the braces. The floor was at least 50 ft. above the water. It was the first bridge built in America with spans greater than could be negotiated with single stringer sticks; and, therefore, richly deserved the fame accorded to it.

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[20] "American Railroad Bridges," *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 1.

The building of this first bridge over the river was noted far and wide. It was considered a remarkable feat of construction. It was reported in many newspapers[21] and was hailed as opening the Vermont region to the trade of Boston and the coast towns of New England generally. It was visited by prominent men. The Rev. Timothy Dwight, President of Yale College, speaks admiringly of it in several of the narratives of his trips up the river. Without doubt the successful building and use of this bridge inspired the building of others in various sections during the next ten or fifteen years.

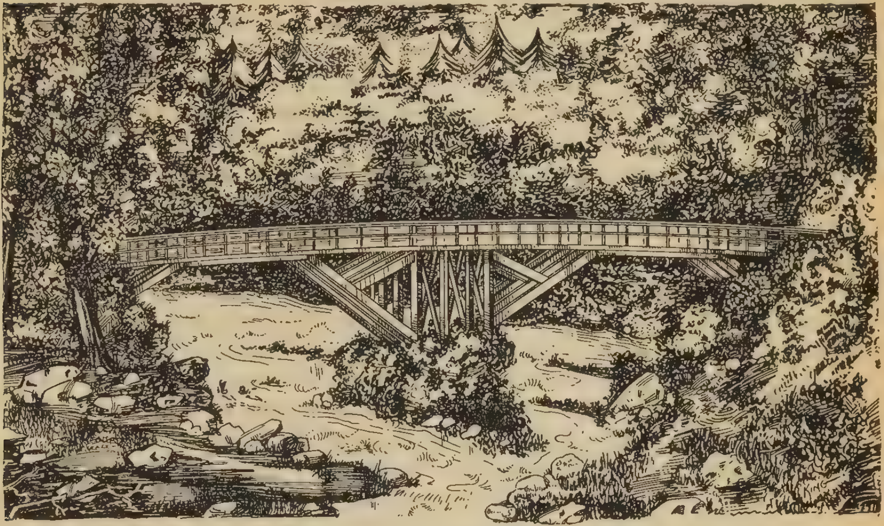
FIG. 8.—ENOCH HALE'S BRIDGE, BELLOWS FALLS, VT.

TIMOTHY PALMER AND HIS BRIDGES

One of the most ingenious and famous of the bridge builders of this period was Timothy Palmer, of Newburyport, Mass., greatly distinguished as a civil engineer (1751-1821). He may well be called the Nestor of American bridge builders. The principal bridges on the Kennebec, Piscataqua, Merrimac, and Connecticut Rivers were built on his models. Mr. Cooper gives a view of the Essex-Merrimac Bridge, built by Mr. Palmer, three miles above

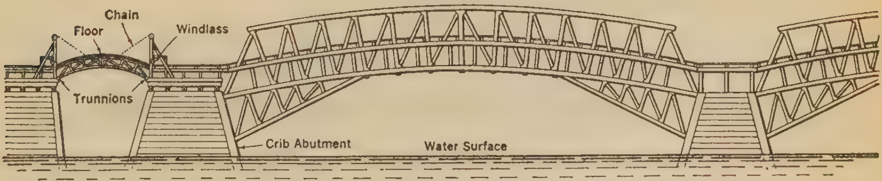

FIG. 9.—ARCH BRIDGE BUILT AT HAVERHILL, MASS., IN 1794.

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[21] *Massachusetts Spy*, February 10, 1785.

Newburyport in 1792.[22] It consisted of two arched spans, one on each side of Deer Island. The Piscataqua Bridge, built in 1794, had pile trestle approaches to an arch which was esteemed a wonder at the time, and the Haverhill Bridge, built the same year, has three arches so similar that Fig. 9[23] will serve to illustrate all three bridges. Mr. Cooper, quoting from a contemporary, describes[24] the Piscataqua Bridge ("The Great Arch") as a masterly piece of architecture. Its length was 244 ft., the rise was 27 ft. 4 in., and the depth of framework of the arch, 18 ft. 3 in. There were three concentric ribs, the middle one carrying the floor of the bridge. The ribs were made from crooked timbers, so that the fibers were nearly in the direction of the curves, and they were connected by pieces of hard and incompressible wood, with wedges driven between. The ribs were mortised to receive these connecting pieces and wedges, thus keeping an equal and parallel distance between them. Each rib was formed of two pieces, about 15 ft. long, laid side by side in such a manner as to break joints. Their ends all abutted with square joint against each other, and were neither scarfed nor mortised, the two pieces of timber being held together by transverse keys and joints. All the timbers were admirably jointed and freely exposed to the action of the air. Any piece might be removed for replacement without injury to the remainder of the structure. Another account states that, of the three ribs or tiers of girders forming the framed arch, the second or middle one supported the roadway, which was 33 ft. wide, and the third served for a railing.

An inspection of Fig. 9 shows a general conformity to the description of "The Great Arch." The short span of 30 ft. at the left is a draw-bridge over the channel. A feature of these bridges that should be noted, is the long braces at the ends, reaching from points half-way down on the abutments to points three-panel lengths forward on the truss.

Ten years later, in his "Permanent Bridge" (Fig. 10), built over the Schuylkill River (1804-1806), at Philadelphia, Pa., Mr. Palmer changed the type radically. The three spans, two of 150 ft. and one of 195 ft., are continuous over the piers, and there is no bracing beneath the arches. The floor is supported by a heavy continuous string-piece notched into the uprights of

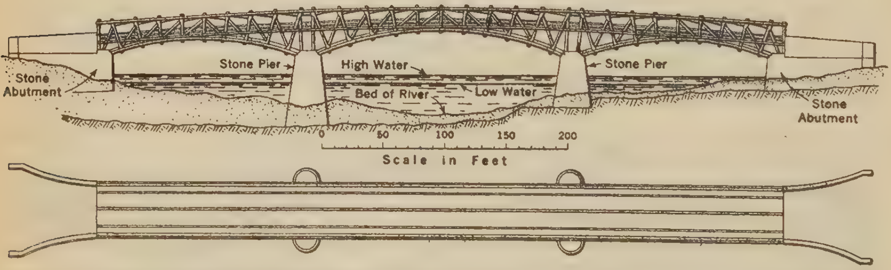

FIG. 10.—THE "PERMANENT" BRIDGE OVER THE SCHUYLKILL RIVER, AT PHILADELPHIA, PA.

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[22] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 4, Pl. I.

[23] This was copied from a pen-and-ink sketch made in 1797, now bound with some manuscripts in Boston (Mass.), Public Library.

[24] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 3.

the continuous arched truss. The large timber available allowed deep notching and the tension members required projecting ends sufficient to resist shearing. Treenails were used instead of bolts and the timber was the best selected white pine. Meantime, in 1796, Mr. Palmer built a bridge over the Potomac River, at Georgetown, D. C., and secured a patent "for his improvement and construction of bridges." He insisted upon the necessity of protecting bridges from the weather, modestly stating that such precaution might extend the life of the bridge from ten to twelve years to as many as forty years. Whether his were the first of the covered bridges is not known.

His bridge over the Delaware River, at Easton, Pa. (Fig. 11), requires special notice as indicating Mr. Palmer's under-estimate of the durability of his work. The obvious changes from the type of the Schuylkill Bridge are the level floor, the straight upper chord, and the flatter arch. The details to be noted are the slotted bolts, the keys, and wedges. Treenails (commonly pronounced "trunnels") are in evidence here, also. A recent report[35] states that it was opened as a toll bridge in 1806 and completed a year later, at a total cost of $61 000. It was always covered from the weather, and was removed in 1895, not because of decay, but because it became inadequate for the traffic when horse-cars were replaced by trolley cars. In other words, it was strong enough, after ninety years, for the loads it was designed to sustain.

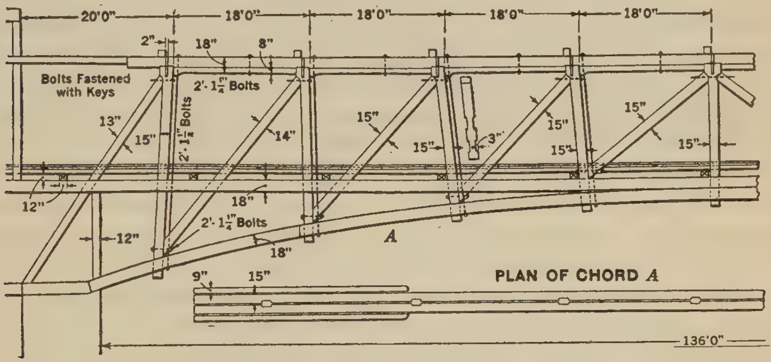

Fig. 11.—Palmer's Bridge Over the Delaware River, at Easton, Pa. (1807).

In Fig. 11, attention should be directed to the style of connections. The bolts at the top of the posts are slotted and held in place by wedge-shaped keys or "fids." These bolts can be driven like drift-bolts in holes of the same size as the bolts, and can be "set up" or drawn in, as the timber shrinks, by re-driving the "fids," or inserting a washer if necessary. Hence, they serve both as treenails and clamps, while a drift-bolt alone serves only as a treenail, and a screw-bolt only as a clamp.

The Second Connecticut River Bridge.—Little more than a decade after Colonel Hale's bridge, a second bridge was built across the Connecticut River which was to be a link on the projected highway connecting Boston with

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[35] Letter from State Highway Dept., Trenton, N. J., March, 1931. Information from Department records, through Roy Mullins, Highway Engr.

Northern Vermont. It joined the Towns of Hanover, N. H., and Norwich, Vt. Dartmouth College was influential in promoting the project. Professor J. K. Lord is authority for the information that[26] in 1796 one, Rufus Graves, built a bridge consisting of a single arch of 236-ft. span (similar to Fig. 9) at this point. The over-all length of the bridge was 344 ft., its width, 36 ft., and its stone abutments, 40 ft. square. It was arched to such a degree that the roadway at the center of the bridge was about 20 ft. higher than at the ends, presenting thus a sharp ascent on entering and a corresponding downward pitch on leaving. It was built of the largest selected pines, 60 ft. long, many of them hewed 18 in. square. The bridge fell by its own weight in 1804. On the testimony given by President Dwight, of Yale College, and others, it seems to be quite probable that Graves' bridge was patterned after the Piscataqua arch, but the builder lacked Palmer's experience and skill.

Lewis Wernwag and His Bridges

Lewis Wernwag was born in Germany in 1770. He died at Harpers Ferry, Va., in 1843. A letter written by his son, John,[27] to the late Samuel L. Smedley, M. Am. Soc. C. E., includes a list of twenty-nine bridges that Wernwag built during the twenty-seven years of his active career in Pennsylvania, Maryland, Virginia, Kentucky, Ohio, and Delaware. He was an original genius. His most noteworthy accomplishment was his "Colossus," built in 1812 over the Schuylkill River at the then, "Upper Ferry," later, Fairmount, Pa. This bridge, with a clear span of 340 ft. was composed of five parallel arched trusses, each with a rise of 20 ft. The roadway, however, had less than one-half that rise. Apparently, he originated the type. This bold design, scientific and architecturally beautiful, probably was never surpassed in America. One Swiss bridge has been mentioned, with a longer span by 50 ft., but compared to this, it was a monstrosity without merit. The cross-section shows the two roadways and two footwalks.

Wernwag did not take out a patent until 1829 and his drawing[27] shows iron rods in every panel, which were intended to act partly as counters and also to adjust the joints if they worked loose. The comparatively light traffic loads of those days would cause but little reaction against the dead weight of such a heavy bridge. The arch was built up in seven layers, and had a total depth of 3 ft. 6 in., and a width of 13 in. Fig. 12 is from the patent drawing and Fig. 13[28] is a full span view of the bridge.

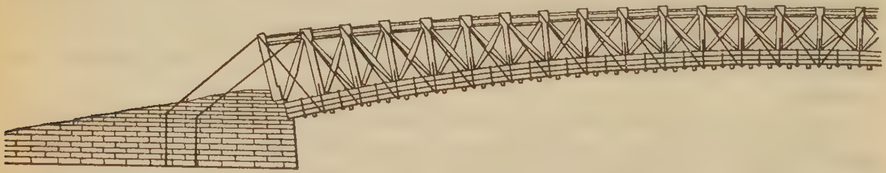

Fig. 12.—Longitudinal Section, The Upper Schuylkill River Bridge, Philadelphia, Pa.; "The Colossus."

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[26] "History of Dartmouth College," by J. K. Lord, p. 657.

[27] *Engineering News*, Vol. 14, (1885), p. 99.

[28] Copied from a large atlas of "Notable Bridges," pub. in London, England, in 1825.

In describing the bridge, Wernwag himself states that dry-rot was prevented by sawing the timbers through the heart to discover defects. The timbers were kept apart by iron links and screw-bolts and were neither mortised nor tenoned, except at the king-post and truss ties. No timbers were allowed to come in contact, and provision was made to tighten necessary bolts as the timber tended to shrink, and any piece could be replaced without injuring the superstructure. The trusses were stayed at the ends by two long iron rods, one attached at the top of the end-post, the other at the top of the second post. Both extended downward at an angle of about 45° and were anchored at the back of a long and massive abutment built with squared-dimension stone laid in parallel courses.

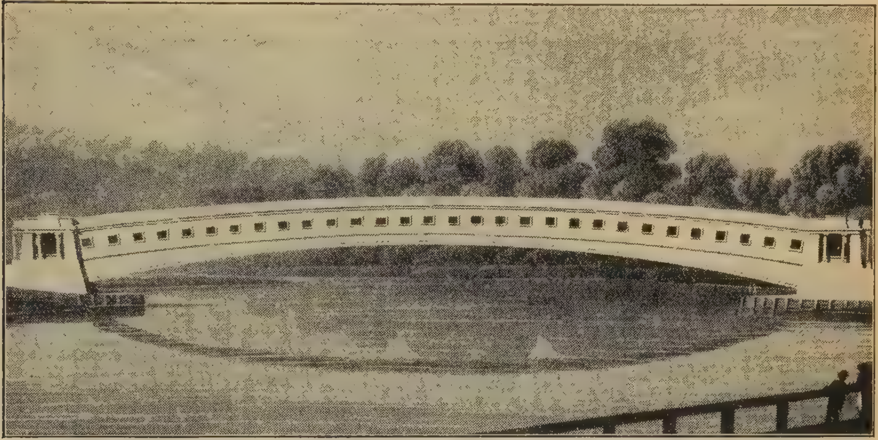

FIG. 13.—THE "COLOSSUS" OVER THE SCHUYLKILL RIVER, AT PHILADELPHIA, PA. (1812).

In 1813-14, Wernwag built a bridge of quite a different type over the Delaware River, at New Hope, Pa. This had trusses with parallel chords, vertical timber posts, and iron rods for diagonals, anticipating what was known later as the Pratt type. The principal feature was the arches which rested on top of the abutments. In other respects, it was based on the same principle as "The Colossus" (Fig. 13) with the addition of the floor chord connected with the casting in which the arch rested, which prevented it from shoving against the pier or abutment.

In 1810 Wernwag constructed a bridge over Frankfort Creek between New York, N. Y., and Philadelphia, which was named "Economy." It was a cantilever[29] type designed so that the center panel could be tipped up in order to pass masted vessels through.

THE BURR TRUSS

Many famous bridges of various designs were built by Theodore Burr, of Massachusetts, during the first two decades of the Nineteenth Century. Fig. 14 illustrates a truss bridge built for railroad service over the White

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[29] *Engineering News*, Vol. 14, 1885, p. 99.

River, at White River Junction, Vt. The photograph was taken when the bridge was being removed in 1890, to be replaced by an iron bridge. It was a splendid example of the Burr type, built in 1848 of old growth white pine when large sticks were available; the end-posts were 18 in. square. The hanger rods were inserted after the bridge was built, to assist in carrying the floor load to the arches as engine loads became heavier. The masonry of this bridge was all quarry-faced granite ashlar laid with very close joints without mortar. At the right end is shown a corner of the north abutment. The abutment wings are circular arcs tangent at the face of the abutment, to a line parallel to the track, presenting the resistance of an arch to the pressure of the earth-fill. The bridge was as strong and serviceable after fifty-four years as when it was built. The entire combination—substructure and superstructure—was capable of much longer service than it rendered. Mr. Burr's first bridge was erected in 1804 over the Hudson River between Waterford and Lansingburgh, N. Y. (Fig. 15). This was followed by a bridge over the Delaware River, at Trenton, N. J., completed in 1806, which carried two roadways and two sidewalks by five arches;[30] the Mohawk River Bridge (1808), at Schenectady, N. Y., which Mr. Cooper

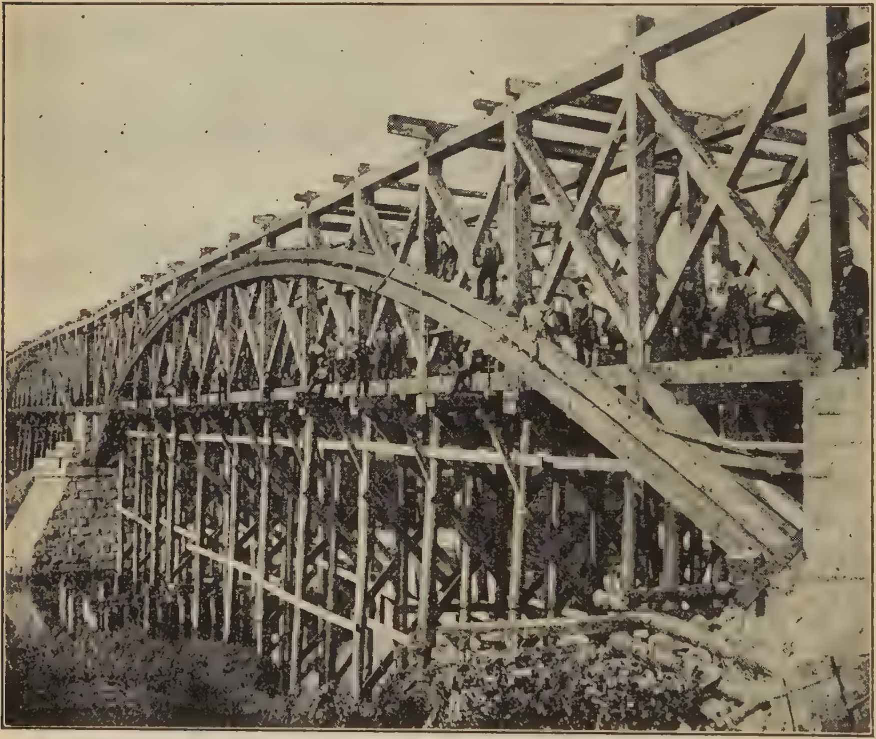

FIG. 14.—BURR BRIDGE OVER THE WHITE RIVER, WHITE RIVER JUNCTION, VT. (1848).

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[30] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), Pl. VII.

described[31] as a wooden suspension bridge; and the long bridge over the Susquehanna River, at Harrisburg, Pa., a wooden arch bridge of twelve spans completed in 1816, on a radically different plan from that of the Trenton Bridge.

These four bridge types seem to have been experimental, in a sense, with Mr. Burr. His patent, dated 1817, which included a drawing of the Waterford Bridge type shown in Fig. 15, ultimately claimed nothing but the arch combined with the truss, the arches springing from the face of the abutment below the bridge seat. Although this bridge was built a year or two earlier, it looks as if it might be a derivation and improvement of Palmer's Easton Bridge (Fig. 11). The parallel chords of the trusses made ideal supports for the roof and floor, its posts and other members were firmly connected with the arches at each panel point, thereby acting perfectly as stiffeners for the arches. The bridge presents a combination of features the theoretical value of which was little realized by Mr. Burr. The other types, built with splendid workmanship, did commendable service, except the Mohawk River anomaly which had to be held up by a trestle after twenty years of use. The Waterford type, under the name of the Burr truss, was used all over the United States as long as purely wooden trusses were built. The type is well adapted for deck bridges, but few prominent instances of such use have come to the notice of the writers. In this type, Mr. Burr really inaugurated the combined arch and truss which has developed in Twentieth Century steel bridge practice, through the spandril-braced arch into the great braced arches at Hell Gate and Kill van Kull, in New York Harbor, and at Sydney, New South Wales, Australia. What are they but two-ribbed arches with stiffening trusses built between?

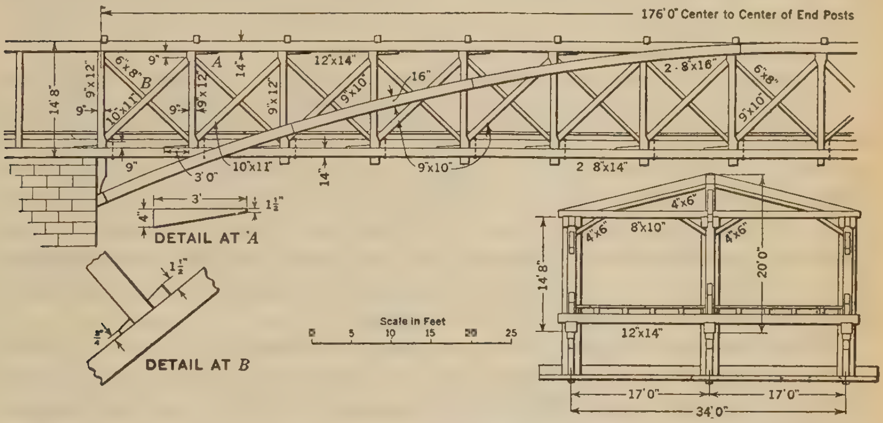

FIG. 15.—HIGHWAY BRIDGE OVER THE HUDSON RIVER BETWEEN WATERFORD AND LANSINGBURGH, N. Y., BUILT BY THEODORE BURR.

The Waterford Bridge, previously mentioned, consisted of four spans of from 154 to 180 ft. clear. It did duty for 105 years without material strengthening until 1909, when it was destroyed by fire. The large timber available and the relatively light loads on bridges in this period of dirt roads rendered these bridges appropriate for the time, although they were frequently designed quite unscientifically. The posts were hewn, not only to make seats for the main diagonals, and for bearings on the arches, but were notched at the lower chord intersections. They acted in tension and had to be framed accordingly. The arches were 4 to 8 in. thick and were attached to each side of each truss. In case of unusually long spans concentric arches were used.[31a] Hundreds of highway bridges framed more or less (generally less), on the Burr principle, were built in various parts of the Eastern and Middle States and in New England. Many of these are still standing. Their longevity is remarkable, if properly protected by roof and side covering. The siding should leave a vertical space at the top about equal to the overhang of the roof; it should be vertical and nailed at the bottom to a girth furred out from the chord to leave a space so that dirt will not collect on the chord. Spans longer than 50 ft. should be reinforced by arches.

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[31] *Transactions,* Am. Soc. C. E., Vol. XXI (July, 1889), Pl. VIII.

The timber in these early bridges was generally white pine which was a factor in their longevity. One at Springfield, Mass., was 104 years old when removed. Two over the Ammonoosuc River, one at Woodsville and the other at Bath, N. H., built in 1828, are still standing (1932). One at Greenfield, Mass., built in 1840, carrying all the traffic from the south, bound for the Mohawk Trail, survived in good condition until June, 1932, giving way then to a new bridge on a revised location. Many of the picturesque old covered bridges still dotting the New England landscape are of the Burr type, most of the shorter ones being without arches.

The longest single span of framed wooden bridge (262 ft.), since Wernwag's "Colossus," of which the writers have knowledge, crossed the Delaware River at Narrowsburg, N. Y. (1859). A system of struts between the posts below the upper chord to resist the horizontal thrust of the main braces, was an unusual feature in a Burr truss.[32] Attention is called to the unusual application of four separate and parallel arches in each of the trusses.

The Town Lattice Truss

Ithiel Town, of New Haven, Conn. (1784-1844), was granted a patent on a plank-lattice design for bridge trusses in January, 1820. It represents a lattice web, more suitable for a fence than for a bridge truss. Town secured a patent in 1835 for double webs and secondary chords.

A double-web truss of this type, with secondary chords, is shown in the paper by Mr. Cooper,[33] but it lacks much in the line of bracing and end support for the short lattice. It shows an unusual and unnecessary prolongation of truss over the abutment. It is true that this type requires a wider seat on the masonry than the Howe and Pratt types, in which the entire reaction is concentrated at a single panel point; but the additional masonry required for the lattice type is negligible if the parapet and wings are adjusted properly. Mr. Cooper, and other writers, state that these lattice-truss bridges are unsatisfactory on account of warping and getting out of line. This cannot be wondered at if lower lateral and effective transverse bracing are omitted, as is the case with the drawings usually shown. The scant justice done this type of truss by writers heretofore, is the warrant for recording its merits and according it considerable space in this paper.

[31a] See Fig. 14.

[32] *Engineering News*, Vol. 42, 1899, p. 122.

[33] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), Pl. X.

Railroad freight traffic in the United States has become so enormously heavy that its economic handling requires equipment so ponderous that wooden truss bridges have become obsolete, even on branch lines, because rural industries must transport their supplies and products in the same equipment that is used on the main lines. On account of the common neglect of rural highway bridges it is generally advisable to recommend concrete structures where facilities for assembling materials therefor are reasonable; but where suitable lumber for lattice bridges is abundant and where fabricated steel is costly, as was the case in New England during the Nineteenth Century, and is now the case in Southern Alaska, Northern Russia, and Siberia, and perhaps in some parts of South America, lattice-truss bridges are economical and can be made perfectly good up to spans of 200 ft. for railroad trains not exceeding Cooper's E-40 loading. If properly covered and maintained, such bridges will give from 50 to 100 years' service.

As stated by Mr. Cooper, this truss has been built in many parts of the United States for railroads where suitable timber was plentiful and cheap. It has also been extensively used for highways before and since the advent of railroads in the Valleys of the Connecticut and Merrimac Rivers, in New Hampshire and Vermont. This and modifications of the Burr truss were the favorite types in building the picturesque old covered highway bridges, so many of which dotted the New England landscape and many of which still remain.

In 1900, at least 100 bridges of this type were extant on the Boston and Maine Railroad System.[34] They had been built largely on the "try-and-fail" method by local carpenters who adopted bridge building as their life-work. The best of these bridges, although built without any knowledge of unit strains, were quite satisfactory under the trains then using them. Common defects were lack of effective horizontal and transverse bracing, inadequate splicing of tension chord planks, and unsafe floors under derailed cars. Notwithstanding these defects, due to lack of scientific design, it is remarkable how well the "try-and-fail" method served, in the hands of the naturally competent mechanics of the Nineteenth Century, to produce bridges adapted to the loads and speeds using them. The excellent workmanship of Timothy Palmer's forces in building the "Permanent" bridge (Fig. 10) is typical of the work done by the early builders of these bridges.

Later practice on the Boston and Maine Railroad developed trusses designed more scientifically to overcome the lack of lateral and transverse bracing in the older bridges, as well as to provide for increasing weight of rolling stock. Fig. 16 shows details that have served to keep these structures straight and upright for thirty years and more, although subjected to loads far heavier than those for which they were designed.

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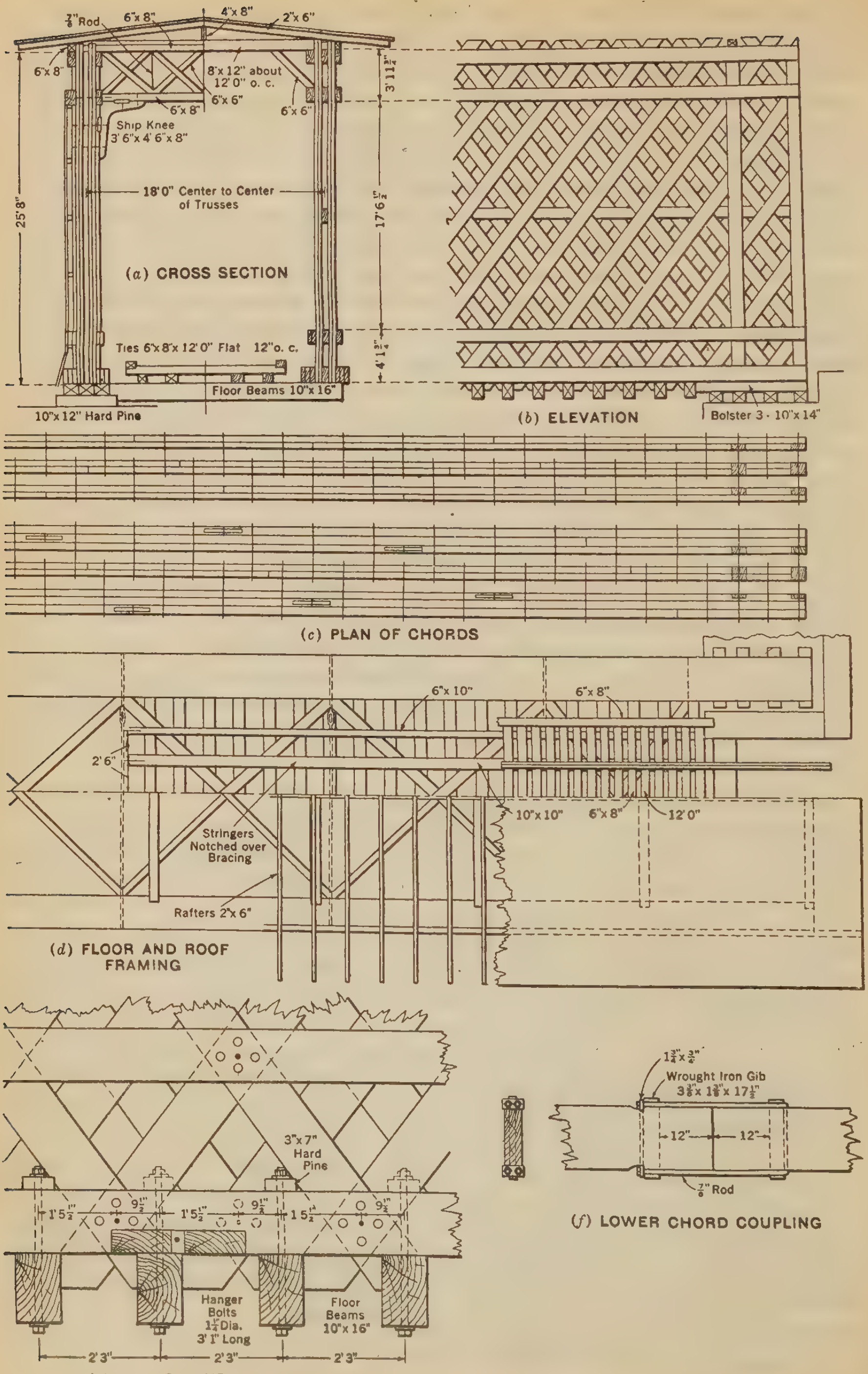
[34] *Journal,* Assoc. of Eng. Societies, Vol. XV, 1895, p. 31.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4"x 8"
2"x 6"
6"x 8"
8"x 12" about 12' 0" o. c.
6"x 6"
6"x 8"
6"x 6"
3' 11$\frac{3}{4}$"
Ship Knee 3' 6"x 4' 6"x 8"
18' 0" Center to Center of Trusses
25' 8"
17' 6$\frac{1}{2}$"
Ties 6"x 8"x 12' 0" Flat 12" o. c.
4' 1$\frac{3}{4}$"
Floor Beams 10"x 16"
10"x 12" Hard Pine
Bolster 3 - 10"x 14"
6"x 10"
6"x 8"
2' 6"
10"x 10"
6"x 8"
12' 0"
Stringers Notched over Bracing
Rafters 2"x 6"
3"x 7" Hard Pine
1' 5$\frac{1}{2}$"
9$\frac{1}{2}$"
1' 5$\frac{1}{2}$"
9$\frac{1}{2}$"
1' 5$\frac{1}{2}$"
9$\frac{1}{2}$"
Hanger Bolts 1$\frac{1}{4}$" Dia. 3' 1" Long
Floor Beams 10"x 16"
2' 3"
2' 3"
2' 3"
1$\frac{3}{4}$"x $\frac{3}{8}$"
Wrought Iron Gib 3$\frac{3}{8}$"x 1$\frac{3}{8}$"x 17$\frac{1}{2}$"
12"
12"
$\frac{7}{8}$" Rod

(*a*) CROSS SECTION

(*b*) ELEVATION

(*c*) PLAN OF CHORDS

(*d*) FLOOR AND ROOF FRAMING

(*e*) LATTICE AND FLOOR DETAILS

(*f*) LOWER CHORD COUPLING

Fig. 16.—Town Lattice Truss.

The distinctive characteristics of the Town lattice are its use of simple sizes of lumber, the small amount of framing required, and the need of nothing but bolts and a few round rods for metal work. In such regions as those mentioned, far from centers of skilled labor and steel fabrication shops, the simple character of materials and labor required by this type is important. The trusses are generally built of uniform sections throughout. While this is a feature that is often criticized as wasteful of material, such waste is much more than balanced by the resulting simplicity of framing and erection.

The Town lattice principle is similar to that of the English iron riveted lattice. Both will stand more abuse from service than any other type of truss. Both will give indications of distress long before collapse, and those that were properly built are found doing duty far longer than many other types. The most successful early builders of wooden bridges placed much emphasis upon the necessity of protection from the weather by roof and side covering. All later experience has proved that bridges not so protected deteriorate rapidly.

Until about 1890, lattice bridges could be built with spruce lumber (then costing about $18 per 1 000 ft. B. M.) for one-half the cost of iron bridges. Twenty years after that date (1910), steel bridges could be built as cheaply as those of wood; and now (1931) steel construction is much the cheaper. Computation for designs of this type is simple, following the methods commonly used for computing plate girders.

The accepted construction and a few important details are shown in Fig. 16. The timber ordinarily used in New England has been spruce, with the exception that for the longer spans Southern long-leaf pine was used for lower chords. The pins were of oak, 2 in. in diameter; and it was considered good practice to soak them in lubricating oil. Steel pipes are now being used in British Columbia for these connections. The stringers are gained over the lower lateral braces and rods. The upper laterals are 6 by 6-in. braces and $1\frac{1}{8}$-in. rods in nearly square Howe truss panels, with independent cross-beams and a transverse tenoned brace near each panel point. Turnbuckles in all lateral rods assist in keeping the bridge in order. The floor-beams are spaced to cut the ends of the tension diagonals less than those in compression. To secure sufficient strength in the lower chords of bridges without arches it has been found advisable in some cases to use iron couplings, as shown in Fig. 16. These are computed to add 16 900 lb. to the section if built as shown.

Noteworthy Bridge Builders.—Mr. Town was a promoter and salesman rather than a builder like his predecessors Palmer, Wernwag, Burr, etc. He sold rights to build his design, published advertising pamphlets, made a trip to Europe, and exhibited business ability of high order.

Sanford Granger (1796-1882), of Chesterfield, N. H., established his home at Bellows Falls, Vt., and became a contractor for bridge and building work. There is a tradition that he patented the lattice type of bridges, but it is more probable that he acquired territorial rights from Ithiel Town to build them. He organized a crew for building this type of bridge and constructed many in Southwestern New Hampshire and Eastern Vermont.

Nicholas Powers (1817-1897), of Clarendon, Vt., during the middle of the Nineteenth Century built many bridges over a wide section of country from Maine to Maryland. His was the famous Blenheim, over Schoharie Creek, in New York State, described subsequently. As Superintendent of Construction he built the great railroad bridge at Havre de Grace, Md., over the Susquehanna River; and as Designer and Builder, many of the covered bridges in Vermont and vicinity were due to him.

Dutton Woods (1809-1884), born in Henniker, N. H., adopted bridge building as a life work in 1837 when railroad construction began to be active. He erected many bridges of various types as desired for several railroad companies from New Haven, Conn., to Concord, N. H., and farther north, as well as many highway bridges for various towns and cities. His favorite type in later years was the Town lattice. He used plank arches where needed. He held the position of Superintendent of Bridges on the Concord Railroad until his death.

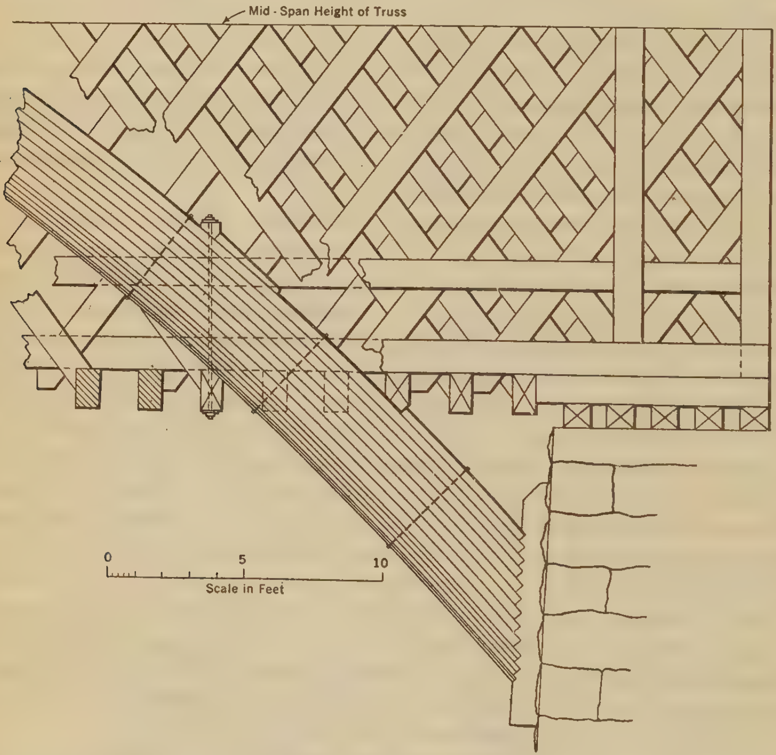


FIG. 17.—TOWN LATTICE TRUSS, WITH BUILT-IN ARCH.

The Childs Brothers, also of Henniker, N. H.—Horace, Enoch L., and Warren S.—built many bridges throughout New England both for railroads and highways, often in conjunction with Dutton Woods. Horace was the carpenter, Warren the stone mason, and Enoch, the Yale graduate and architect-engineer.

David Haselton, born in Dorchester, N. H., in 1832, deserves mention as a builder of lattice bridges after 1860. He was Superintendent of Bridges and Buildings on the Boston and Lowell Railroad until 1890, when it was leased to the Boston and Maine.

Supplemental Arches.—Spans of more than 80 ft., depending on the load, should be built as combined truss and arch. Ordinarily, a single arch on the center line of the truss is sufficient, but, if desired, an arch may be built on each side of the truss, Burr fashion. These arches spring from skew-back blocks scribed to fit the masonry below the bridge seat. The arch planks are firmly held together by radial bolts, and the load is applied by vertical suspension rods carrying needle-beams below the lower chords, or by passing through the floor-beams if they are underhung. These suspenders (or posts when over that part of the arch near the end which is below the lower chord) should be not more than 10 ft. apart along the arch to prevent it from buckling on account of its laminated structure. Fig. 17 shows this construction.

Combinations of arch and truss have been disapproved by schoolmen because of the impossibility of determining the theoretical distribution of stress between the truss and the arch; but if there is strength enough in both, and they are properly yoked together, the safe elasticity of the wood and the connections will take care of the distribution. The Town lattice is the best of the various types of wooden trusses to serve as an arch stiffener because its web members serve either as counters or main braces. The value of arches framed with trusses, if heeled against the masonry, has been amply demonstrated by other types of trusses, some of which have stood for a century. If these arches are sprung from the lower chord the purpose of using them is defeated.

Highway Bridges.—Hundreds of highway bridges have been built with lattice trusses, mostly with single lattice; that is, with one web system per truss. Fig. 18 shows such a bridge of two spans built in 1840 and removed in 1930 to be replaced by a two-span concrete arch structure. This bridge replaced the successor of the famous Enoch Hale Bridge (Fig. 8), and was on the identical site, about 15 ft. higher than the old one. It failed from neglect of a decayed wall-block which led to irreparable injury to one truss. It was 262 ft. long over all. Tolls were eliminated in 1904.

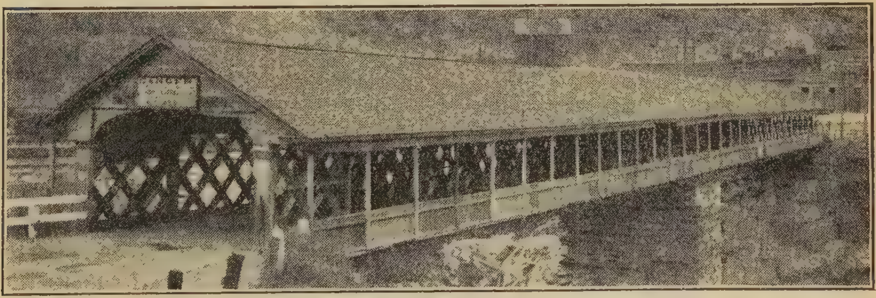


FIG. 18.—TUCKER BRIDGE, BELLOWS FALLS, VT. (1840).

The Timber Lattice.—The type of truss shown by Fig. 19 has been used extensively for highways in the Connecticut River Valley and especially in Vermont, and, in a few instances, for railroad bridges. It is not known who first adopted it,[34a] but its merits have been abundantly proved. The arches shown below the chord were added in 1927 to relieve a decayed lower chord.

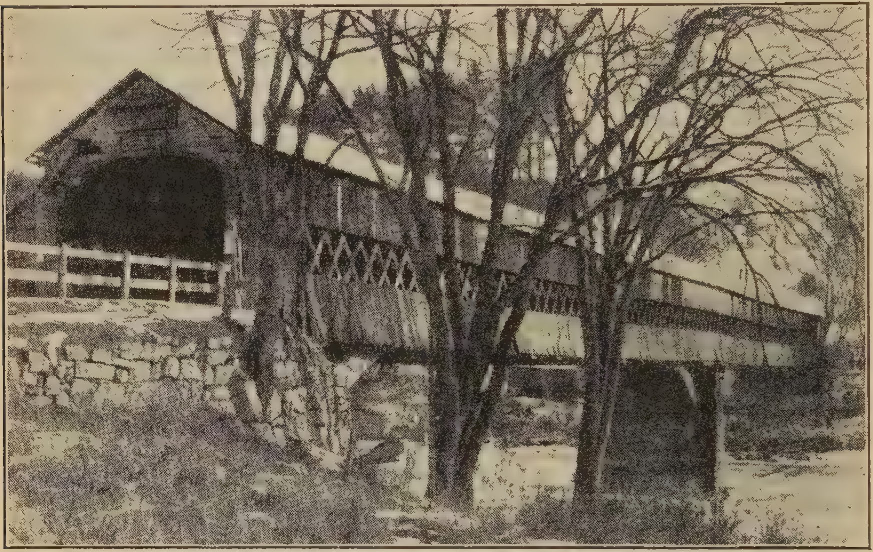


Fig. 19.—The Ledyard Bridge, Hanover, N. H. (1859).

Several highway bridges of this type built 60 or 70 years ago are now standing, some with spans of 160 to 180 ft., over the Connecticut River. The most common reason for unsatisfactory conditions in this type is decay of some part not effectively protected or not renewed in time. In common with plank lattice of their time, built without knowledge of unit strains, they are frequently weak in the tension chord and deplorably deficient in floor-beam strength. (Blenheim Bridge, a different type, shown in Fig. 20, is mentioned subsequently.)

Unlike railroad through bridges, the trusses of highway bridges are not of great height so that they seldom suffer from lack of transverse or lateral bracing. Their plank floors, unlike the open floors of railroad bridges, make the best kind of lateral bracing. The usual construction is a web composed of intersecting joists, 6 by 8 in., or larger, set with broad sides parallel to the plane of the truss, in panels of about 4 ft. which, with trusses of approximately 16 ft., out to out of chords and a run of three panels, make seven crossings of each web member in its length.

At these intersections each member is notched 1¼ in., making the web 9½ in. thick. Where the members pass through the chords, both web members and chord-sticks are notched 1 in. each, leaving 7½ in. of net timber for the web and 5½ in. clear between the sections of the chord. For spans of less than 75 ft., secondary chords are not needed. The ends of the diagonals should be cut not less than 9 in. vertically from the chord, in order to allow a safe length to resist the longitudinal shear.

[34a] Ithiel Town obtained a patent on it in 1839, p. 332.

Pony Trusses.—For spans ranging from 30 to 50 ft., pony trusses have been used extensively. Modified Howe trusses with inclined end posts, Town lattice, and mongrel types are common. The trusses of this class must be stayed by sway-braces. It is not good practice to set these braces on an extended floor-beam because each passage of a load would throw the top of the truss back and forth on account of the deflection of the floor-beam. The needle-beams carrying these braces should be blocked down so that the stringers shall not rest on them.

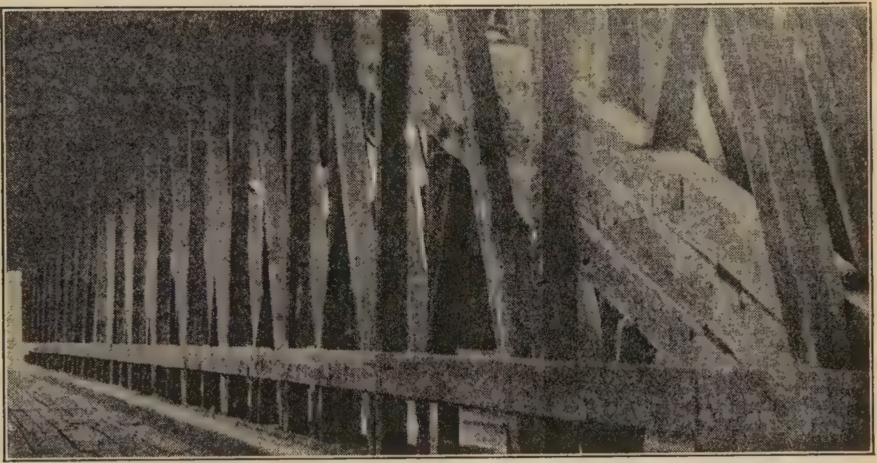
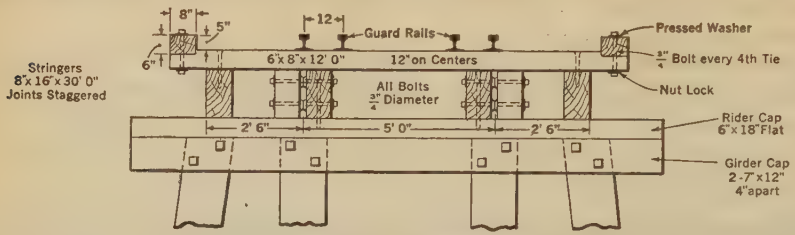


FIG. 20.—BLENHEIM BRIDGE, SCHOHARIE COUNTY, NEW YORK (1855).

Stringer Bridges.—A history of wooden bridges would not be complete without mention of the humble stringer bridge which was, of course, the earliest recourse of man for crossing deep streams, as stated under "The Ancient Background." In American Colonial days just such structures with log cribs or rough stone abutments and round logs hewn on top to receive a plank floor were used for openings up to 40 ft. For wider streams pile or frame trestle bents were used in quiet waters and log cribs filled with stone where heavy freshets occurred. In the days of large timber and of dirt roads this type sufficed for highways, but with railroads, spans of plain stringer bridges were limited to about 15 ft. Fig. 21 shows a type of single-track, pile-stringer bridge of the last decade of the Nineteenth Century.

For spans from 15 ft. to 30 ft., compound keyed stringers have been found useful. Fig. 22 shows the details. The keys shown are cast iron and are proportioned for the longitudinal shear and, hence, the total depth of the compound stick can be used in computing its moment of resistance. They are placed as nearly as may be in accordance with the intensity of the shear. The thin fin at the middle of the key is cut into the wood to hold the key from working out of place.

Fig. 22 shows the junction of a compound and single stringer on a trestle bent. The pile-cap is called a "girder cap with rider"; it is used only on first-class pile work. For lower grade pile work and for framed bents, solid caps, either tenoned or drift-bolted to the pile or post, are used.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5"
6"
12
Guard Rails
Pressed Washer
3/4" Bolt every 4th Tie
Nut Lock
Stringers 8"x16"x30'0" Joints Staggered
6"x8"x12'0"
12" on Centers
All Bolts 3/4" Diameter
2'6"
5'0"
2'6"
Rider Cap 6"x18" Flat
Girder Cap 2-7"x12" 4" apart

FIG. 21.—STANDARD TRESTLE SECTION.

Although it can be said that the day of wooden bridges has passed, it is a fact that many hundreds of miles of wooden trestles, stringer, and truss bridges are still in service in the United States. This brings the discussion down to the simple stringer and trestle, illustrations of which, other than Fig. 21, are scarcely needed.

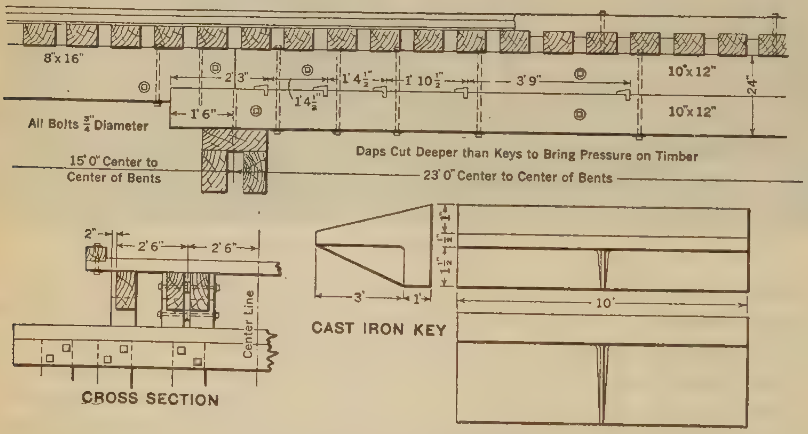
8"x16"
2'3"
1'4½"
1'10½"
3'9"
10"x12"
24"
1'6"
1'4½"
10"x12"
All Bolts 3/4" Diameter
15'0" Center to Center of Bents
Daps Cut Deeper than Keys to Bring Pressure on Timber
23'0" Center to Center of Bents
2"
2'6"
2'6"
Center Line
CROSS SECTION
3'
1'
1½"
1½"
1"
10'
CAST IRON KEY

FIG. 22.—KEYED STRINGER SECTIONS.

LONG'S BRIDGES

In 1830, Bvt. Lt.-Col. Stephen H. Long, U. S. Topographical Engineers, patented a truss which bore his name. He afterward modified it and was granted other patents in 1836 and 1839, "to produce lateral stiffness or inflexibility in wooden or frame bridges." This was effected by a system of lattice bracing applied either to the upper or lower chords, or to both. Concerning Colonel Long's truss Mr. Cooper remarks[35] that this is another step in simplicity and concentration of parts. Here also iron did not enter as a necessary part. Connections were made by framing the parts together or by use of wooden keys or treenails. Many bridges of this form were built, but it was never widely popular. Fig. 23, from a drawing of 1830, shows two sets of braces at the abutments below the lower chord reaching forward to the first and second panel points.

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[35] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), p. 9.

The remark of Mr. Cooper applies in particular to the design of 1830. Colonel Long's later designs departed from the simplicity displayed by the first one. The use of abutment braces extending to the first and second panel points breaks the continuity of the frame as a simple truss, independent or unassisted between the bridge-seats. The term, "assisted truss," may be applied to it. The cogged cast-iron splice-blocks (or plates), which his patent drawing shows, might be used instead of wooden splice-blocks. This idea might have found favor in the days of cast iron, so to speak, but is more worthy of consideration now when steel castings with their greater strength are more readily available.

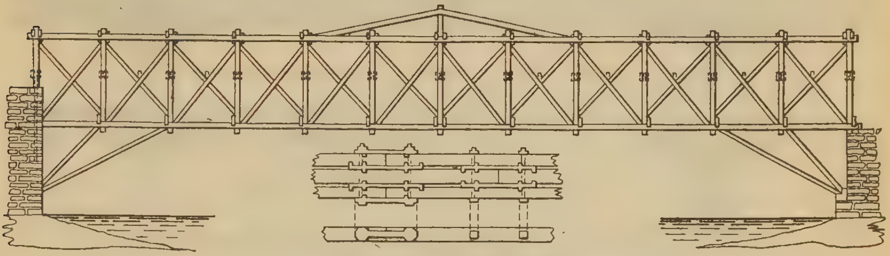

Fig. 23.—Long's Bridge.

Many rural highway bridges were built throughout the Eastern States by local carpenters, which were hybrids of the Long and Burr types, without the science of Long nor the arches of Burr. It is impossible to classify them as of any type but, due to the practical common sense of their builders and the accommodating property of wooden framing to distribute strains, these "mongrel" bridges have often given service equal to some of the widely known patented types.

The Howe Truss

In 1840, a patent was granted to William Howe, of Massachusetts, which was the basis of the Howe truss bridge so generally built throughout the country for half a century and more after the date of the patent. There were two systems of web members; that is, the diagonals ran two panels. The patent drawing also shows a secondary lower chord about 2 ft. or more above the main chord, the only function of which seems to be to carry the horizontal thrust of the end brace of one system of diagonals. It is doubtful whether there was ever a truss built with this arrangement of secondary chord and short end braces. During this season (1840), Mr. Howe, in company with Amasa Stone (1818-1883), built the great bridge over the Connecticut River, at Springfield, Mass., to carry the Western Railroad (later the Boston and Albany), which was opened for traffic the following year. This bridge had seven spans of about 190 ft., center to center of piers. The trusses had two systems of web members like the patent drawing, but the end diagonals of both systems were carried on a single-angle block at the bridge seat. (See Fig. 24.) The lower chord consisted of six sticks, running six panels of about 7 ft. each, packed close together without splices of any kind. The diagonals and rods were of uniform size throughout. In 1874, the bridge was replaced with an iron riveted lattice. It is not known to the writers that there has been another bridge built of the Howe type with webs of two systems. In 1841, Mr. Stone bought the Howe patent, formed a company, and built this type of bridge in many parts of the country.

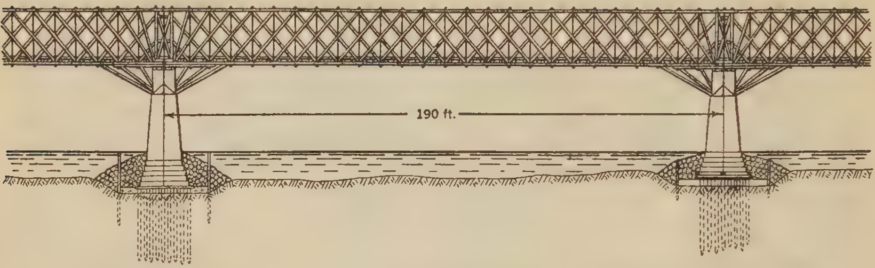

Fig. 24.—Bridge Over the Connecticut River, at Springfield, Mass.

In this type the vertical iron rods are a vital part of the web system; and the possibility of proportioning the web members for the stresses which they transmit, as well as the simplicity of the joint connections through which that transmission is effected, make this a nearly perfect type of truss. Early bridges of this type were built of white pine and, later, of spruce, and, finally, of Southern long leaf pine which, although much heavier than the other species named, is better adapted for the Howe type. The splice-clamps and keys are usually of seasoned oak. Oak angle-blocks were used for thirty years or more, but later practice considered cast iron better than wood to transmit the forces acting at the panel point. In general, Fig. 25 shows, by isometric projection, the arrangement of the members of a Howe truss, deck, railroad bridge built in 1874 over the gorge near Quechee, Vt. By cutting off the members assembled at one panel point the view discloses all the truss members and auxiliaries in the bridge, namely, the upper and lower chords, the web system of the main and the counter braces and the vertical rods, the horizontal lateral braces and rods, the vertical sway-braces, the latticed lateral system nearly the same as that proposed by Colonel Long, wooden angle-blocks, and the common arrangement of splice-clamps.

This bridge had a pair of light arches (which is unusual with this type), taking load from the truss by means of vertical rods which, passing through the needle-beams below, received whatever load the latter transferred to it. A proper splice is shown in the lower right-hand chord, and an imperfect, one-sided splice is shown on the farther chord at *D*. Each chord-stick was spliced in every fourth panel. This bridge had a clear span of about 200 ft. and a height of 24 ft., center to center of chords. There were eighteen panels, 11½ ft., center to center of verticals. The track was about 165 ft. above Ottaquechee Brook, and the bridge was carefully protected by boarding.

More recent practice improved the application of the Howe design by increasing the size of the web members and decreasing the strength of the lower chord-stick connections from the center of the truss to the ends, according to the stresses sustained. Fig. 26 shows a Howe truss substantially as built in 1889 at Morrisville, Vt., which displays these advances over the older practice. The rods in the panels next to the center are two $1\frac{3}{4}$-in. and one $1\frac{1}{4}$-in. and, in the end panel, three $2\frac{3}{8}$-in. and two $2\frac{1}{4}$-in., all upset, with gib-plates varying from 6 by $1\frac{1}{4}$ by 36 in., next to the center, to 14 by $1\frac{7}{8}$ by 50 in., at the ends. The main braces vary from three sticks, 7 by 7 in., at the center, to three sticks, 11 by 13 in., at the ends. The increase in strength of the lower chord-stick connections from end to center can be seen on the plan. Note the filler opposite the single splice-clamp in the fourth panel from the end. The cast-iron angle blocks are all of the size computed for the end panel. This practice leads to excess of iron, but saves expensive pattern-making.

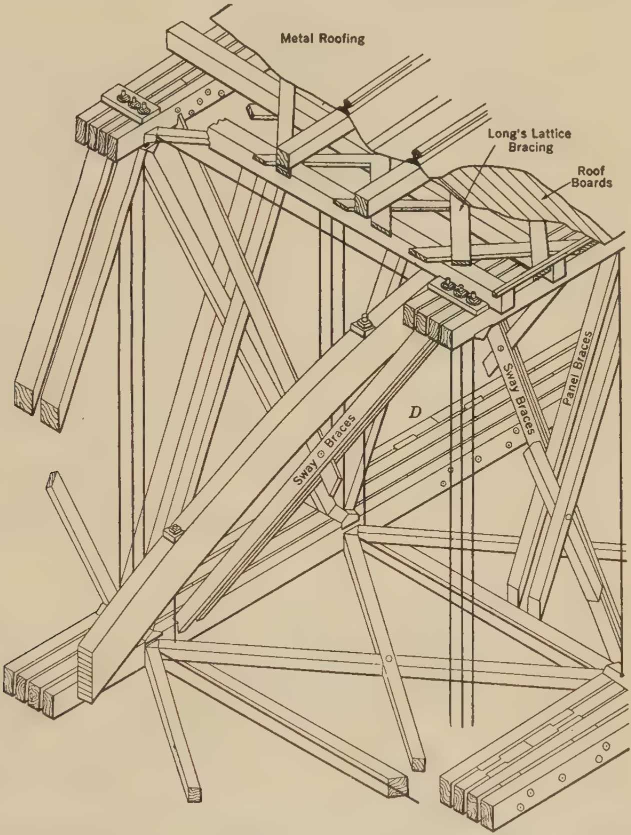

Fig. 25.—Isometric View of Howe Truss Across Ottaquechee Brook near Quechee, Vt.

For through bridges, in order to have sufficiently rigid portal-bracing and to return the side boarding to cover the end panel inside, the upper chords are extended to a connection with the end-posts which may well be in two sets with light rods between them, as shown. Howe truss bridges should be protected from the weather by roofs and side covering, the same as other types of wooden bridges. The lack of this protection has caused the premature failure of many otherwise good bridges, the life of which would have been prolonged two or three times if they had been covered as advised by the early masters of timber framing.

In 1850, a bridge was built for the Sullivan County Railroad Company, at Bellows Falls, Vt., across the Connecticut River, by George A. Parker, of Massachusetts, who, later, was Chief Engineer of the great Havre-de-Grace Bridge over the Susquehanna River in 1862. In the Bellows Falls Bridge were two 250-ft. spans of Howe trusses built with an arch on each side of each truss, Burr fashion. This bridge was replaced in 1882 by a riveted iron bridge on the same masonry that carried the wooden bridge. Arches, although sometimes built with Howe and Pratt trusses, are not indigenous with them so to speak. The south span was saved by its arches during a great freshet in 1862, when a span of the highway bridge at Charlestown was swept off and came floating down the river erect and whole, striking the railroad bridge "head on." An eye-witness states that, as the lattice-web system of the highway bridge crushed and gave way against the rugged chord of the railroad bridge, the ridge-pole of the "attacking" bridge pierced through the standing bridge, and protruded so far on the down-stream side that, as soon as the wreckage had quieted enough, men ventured out on the bridge and sawed off the overhanging stick for fear it would twist the truss. The bridge was crowded a few inches out of place on the abutment. No further damage was done.

A somewhat similar case, showing the usefulness of arches, occurred at Penacook, N. H., where a lattice bridge with an arch built on the center line of each truss (see Fig. 17) spanned the Contoocook River near its mouth. A great jam of ice, during a winter thaw, piled up against the bridge, several feet above the floor level, and sprung it 2 ft. out of line at the center; but the arches, seated on the face of the abutments, withstood the pressure, and no serious damage occurred. Except for the arches the bridge would have been pushed into the Merrimac River.

During part of the Civil War (until 1866), Mr. Parker was engaged in building the bridge for the Philadelphia, Wilmington, and Baltimore Railroad Company over the Susquehanna River between Perryville and Havre-de-Grace, Md. The superstructure was of the same type and span length as the Bellow Falls Bridge. The writers know of no other Howe truss bridges built with arches in pairs, as in these two.[36] Nicholas Powers, who had worked on the Bellows Falls Bridge, was asked by Mr. Parker to work on that at Havre-de-Grace, under a Mr. Bagley, the Superintendent. When the spans were nearly erected a tornado wrecked them, twisting an approach trestle badly out of line. Mr. Bagley was blamed and discharged. Mr. Powers prepared to go also, but the Manager asked him to take charge of straightening the approach, and finally arranged with him to superintendent the reconstruction. Mr. Powers agreed to do it for $7 per day if, to start with, he could pick fifty men. His terms were agreed to, with the promise of $500 bonus if he finished the bridge without accident. Mr. Powers had his 16-year old son as Assistant, and, at times, had 400 men employed on the work. Meanwhile, the haying season was on at the home farm in Vermont. To the urgent letters from his wife to come home and secure the hay crop, he replied by sending money to hire the needed help. The bridge was finished sometime in November, a celebration was planned for the dedication, and a picture was to be taken of the bridge crew with Mr. Powers in the center; but he had promised his wife that he and their son would be home for Thanksgiving; he kept the promise and the picture was taken without him.

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[36] A good wood engraving of two of the twelve spans of the Havre-de-Grace Bridge resting on stone piers and abutments, may be seen in Appleton's Cyclopedia, Vol. III, p. 273, pub. in 1873.

The great popularity of the Howe truss was well deserved. Many hundreds were built in all sections of the United States and Canada. However, this type cannot be classed strictly with wooden bridges, inasmuch as it uses iron rods for web-tension members—one of the four essential elements of the open framed truss. The development of railroads at the time of Mr. Howe's patent (1840), gave a great impetus to the manufacture of iron, and made it possible to procure round rods in quantity. The type is really intermediate between wooden and iron bridges.

Iron angle-blocks were sometimes cast with tubes extending through the chord, as in Piper's patent of 1867, for the double purpose of furnishing resistance to horizontal thrust by being notched into the chord-sticks; and, also, by contact with the gib-plates, to resist the vertical pull of the rods. Some writers specified that the tubes be made short enough to allow for shrinkage of the chord timbers; but such shrinkage may be more or less than the allowance; and it is best to provide sufficient hold on the chord itself for the horizontal thrust, and also gib-plates large enough to prevent side-grain crushing of the chords. If the tubes and gib-plates come in contact, the chord is generally loose and does not properly support the angle-block. Hence, it is best to omit the tubes and design the angle-blocks and gib-plates to do their part without them. Unless the cast-iron angle blocks are unusually large, it is not profitable to attempt to save metal by coring the central part, although this is the practice of some designers.

Colonel Long's patent of 1830 claims a cast-iron splice plate for the chord-sticks. A modification of this as a steel casting for splicing the chord-sticks in Howe trusses is well worthy of attention. The plate can carry four cogs instead of two, as shown by the wooden splice, and can act as a key by having cogs on both sides and also as a spacer if rests are cast to bring the plate in the center between the spacing of the chord-sticks. The principal difficulty in the use of these plates is in boring the holes if the timber and plates are in place; but it can be done by the skillful use of a ship-auger type of bit (see Fig. 23).

Designing the angle-block of the Howe truss involved the brace-in-panel problem, which is to have the squared ends of the timber brace and the

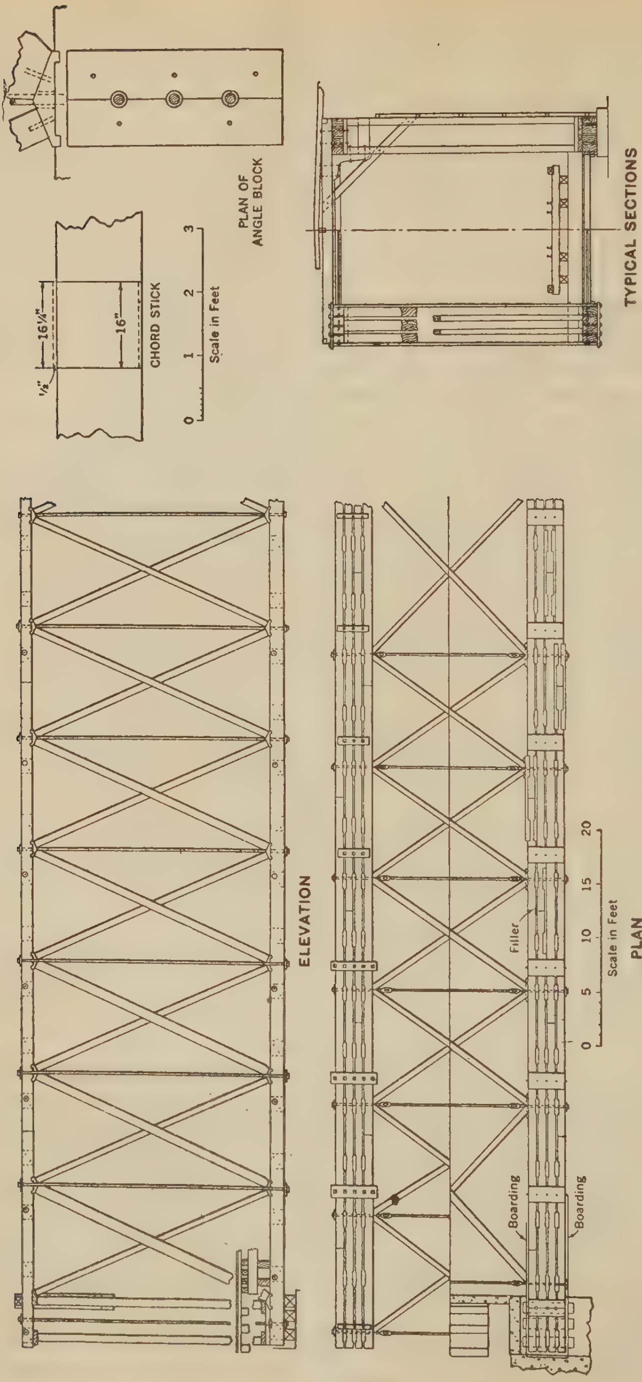
PLAN OF ANGLE BLOCK
½"
16¼"
16"
CHORD STICK
0
1
2
3
Scale in Feet
TYPICAL SECTIONS
ELEVATION
Filler
Boarding
Boarding
0
5
10
15
20
Scale in Feet
PLAN

FIG. 26.—PLAN AND SECTIONS, HOWE TRUSS RAILROAD BRIDGE (1889).

inclined bearing surface of the angle-block fitted so that the center line of the brace and the vertical axis of the rod will meet on the center line of the chord. This involved also the camber to be given to the truss. In theory, it is a problem in higher mathematics. In practice, each builder usually had his own rule, and the best one was to lay out a panel full size and cut a template for the brace, leaving just such an angle-block as he wanted, and then, for camber, cut his counters a little short, the amount determined by his experience. The camber was given partly for the sake of appearance. Optically, to some observers, a truly horizontal chord might appear to have a slight sag. Even an actual slight sag was of no consequence if it did not increase so as to indicate over-strain of the truss. However, there is a good reason for camber, because of the compressibility of wood and the imperfection of hand-made joints.

Keys and joint clamps should be framed with bearing surfaces and notches cut with a slight draft so as to require driving to place when erected (see details in Fig. 26) and the depth of the notches should be such that the end fiber resistance to the compressive strength at the notch should be equal theoretically to the tensile resistance of the uncut cross-section of the necks of the clamps. The notch must be made far enough back from the end of the chord-stick so that the lengthwise shearing resistance of the softer wood shall be so adjusted to the pull that a slab will not split off between the notch and the end. This is usually the weakest part of the connection. Many splices have failed by such splitting. Almost invariably, in old bridges of this type, outside sticks have only their inner side clamped, as shown at *D* in Fig. 25; and all the clamp bolts are in the head of the clamp, leaving the ends of chord-sticks free to bend outward, which increases the tendency to split off at the notch. The builders were very loath to bore through the neck of the clamps.

THE PRATT TRUSS

A patent was granted in April, 1844, to the late Thomas W. Pratt, M. Am. Soc. C. E., and Caleb Pratt, the claims for which are indicated in a drawing of which Fig. 27 is a copy. In this patent the attempt was made to

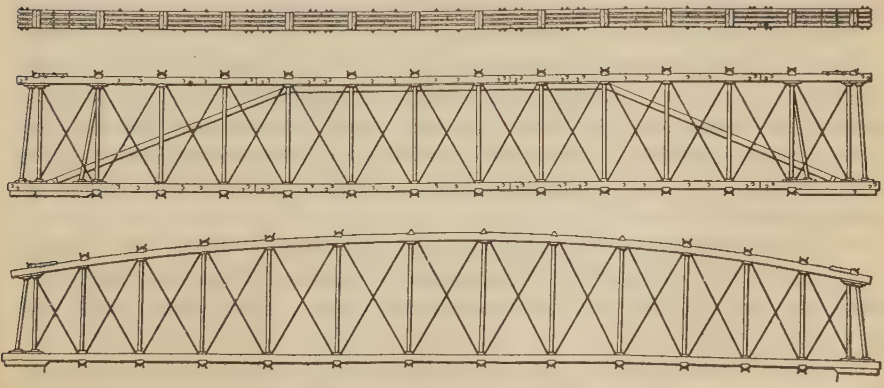

FIG. 27.—PRATT TRUSS, AS PATENTED IN 1844.

use timber in the simplest and most efficient manner in the web members by confining it to the verticals. The advantage of doing this, however, is over-matched by the longer rods required than in the Howe truss, and the awkward angle-blocks needed for these diagonals. For all metal trusses, however, the Pratt truss is ideal.

For railroads, the continuing increase in weight of equipment and loads has kept the problem of strengthening bridges before engineers. For the early Howe and Pratt types it has been found that the tension rods are the weakest single feature. In the Howe truss, yoking on additional rods is an easy remedy; furthermore, reinforcement is easily applied to the timber diagonals. In the Pratt truss, this is far more difficult; but the Pratt truss permits curved upper chords, as shown in the lower drawing, and this is desirable for long spans. Evidently, in the Howe type, curvature of the upper chord would introduce complications in making proper seats for the varying lengths and inclinations of the diagonals.

In the parallel-chord truss the king brace and strain-beam shown in the upper drawing of Fig. 27 is a reversion to the European practice more than a century old in Pratt's time. In a panel truss it befuddles computation of web stresses and is a faulty device on several counts for assisting a truss which should be designed to be strong enough without it.

The activities of the Childs Brothers in building bridges, especially for the New England railroads when these were young, has been noted herein previously. The drawing accompanying the patent granted to Horace Childs in August, 1846, indicates an attempt to combine the Howe and Pratt types. It is defective in omitting counter-rods in the middle panels, and peculiar because of an awkward attachment of the main braces by a joint bolt passing through the chords and the angle-block, and up into the brace, which was a crude method of making the main braces act as counters. The Childs Brothers built many bridges of various types, among them the Granite Street Bridge, in Manchester, N. H., over the Merrimac River, in 1862, and one over the same river in Hooksett, N. H., in 1858.

TOLL BRIDGES

Incidentally, the term, "toll bridge," has been applied to several of the old bridges previously described. Some may not realize that, at first, with possibly a rare exception, all bridges were built by private initiative under charters granted by the State. The charters specified the tolls to be charged, subject to change only by decree of the Court. In populous districts the interruption of travel at the toll-gates became so inconvenient and annoying that counties and towns bought the charter rights and gradually made the bridges "free."

Of thirty bridges built across the Connecticut River between New Hampshire and Vermont, north of Massachusetts, beginning in 1785, twenty-six were chartered by law and operated as toll-bridges. At present (1932), all but two are free.

In this Twentieth Century, comes a reversion to the toll system, or policy. In the great metropolitan districts, and at points where wide streams must be crossed by trunk-line highways, the cost of erecting and maintaining the huge and lengthy structures, with capacity sufficient for the enormous volumes of light and heavy traffic, is too great to be collected by general taxation. It must be retrieved by toll charges upon those who use the facilities.

WOODEN DRAW-BRIDGES

The roads over the broad estuaries and tidal streams mentioned under "American Timber Bridges," invariably required openings for the passage of water craft. In pioneer days, land transport was difficult, and it was desirable to get water-borne traffic as far inland as high tide would float the small vessels then used. Invariably, draw-bridges called "tip-ups" were built in such cases. They were narrow so that four oak logs sufficed for stringers. Crooked trees were selected with large butts which helped as counterweights. These logs were 12 to 15 ft. longer than the width of the channel to be spanned. The stringer portion over the opening was hewn as small as practicable. An oak log firmly framed to the stringers, with rounded ends to turn as trunnions, served the purpose of a hinge. Chain guys passing over sheaves at the top of a gallows frame spanning the roadway, actuated by hand windlasses, raised the free end of the bridge when desired. If the opening was too wide for a single leaf, two were used meeting at the center, with a toggle arrangement to hold them in grade under passing loads.

These bridges required the lifting of a section of the plank over the hinge, to allow the draw stringers to turn up. This is simple in a highway floor; but with the introduction of railroads, difficulty developed in lifting the rails and their supports, so that a swing bridge of the type called the "A-frame swing-draw," was adopted. The stringers of this bridge were framed to rest at the heel—when swinging—on a pivot under the center of the track, at a distance back from the channel cap. The head of the A-frame was vertically over the heel pivot when hauled backward for raising the swing end of the bridge. The guy rods running from the head of the frame to the needle-beam under the stringers were held apart for side clearance of the train by a spreader-beam high enough above the track for proper head clearance. A hauling-off beam attached by a pin to the side of the bridge carried a rack into which a pinion, operated by a hand-crank on shore, meshed to swing the bridge off and on,[37] the same as indicated by the illustrations referred to under "The Jackknife Draw-Bridge."

The Jackknife Draw-Bridge.—The A-frame draw was awkward to arrange for in some locations. When the Gloucester Branch of the Eastern Railroad was built in 1845, a draw-bridge of some other type was required for the entrance of the little harbor basin at Manchester, Mass. Joseph Ross (1822-1903), a young mechanic of Ipswich, Mass., and Samuel Ashburner, a draftsman, engaged on the work, "got their heads together" and designed what was afterward called the jackknife type of draw-bridge. This important development has been adequately described elsewhere.[38]

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[37] *Engineering News-Record,* November 5, 1931, p. 719

[38] *Loc. cit.,* January 21, 1932, Vol. 108, p. 93.

A Unique Bridge

A bridge of singular distinction, the pride of Schoharie County, New York, deserves special mention. The location is North Blenheim, on Schoharie Creek. (See Fig. 20.) Erected and long maintained as a toll bridge, it was finally taken over by the town which has carefully kept it in prime condition. The record states that the "contract to design and erect the bridge was awarded to Nick Powers, a Vermont Yankee who built many such structures," and that "the bridge was not built over the stream as many would believe, but was constructed piece by piece, back of the present village near the site, and there set up to make certain that every part was perfected before erection over the river was attempted." While this was being done, the masons built the abutments. The material is native oak and pine. The unique feature is the single arch, framed and built on the center line of the middle truss, and this truss extends upward 6 or 7 ft. higher than the side trusses, so that its upper chord forms the ridge of the roof. If the floor timbers extended across the entire 26 ft. of the width, thus serving both roadways, they would contribute greatly to the stiffness and stability of the entire structure.

From available evidence it seems likely that the posts and diagonals were framed to pass through the chords as in Long's design. It has the unusual span of 228 ft., but this gives no distinction over bridges already considered. It can scarcely be classified or correlated with any other type of bridge that has come within the knowledge of the writers. Built in 1855, it is claimed to be quite serviceable for the traffic which it now carries after seventy-six years of usefulness; but the County Officials decided that it must be replaced by a modern concrete structure. However, according to later report, the bridge will be maintained as a local antique.

Retrieving Weak and Injured Bridges

It may be said that it is more feasible to strengthen a wooden bridge in a satisfactory manner than one of metal or masonry; and that the deficiency in the strength of a bridge scientifically designed is more difficult to remedy than in one built by guess, when one member is proportionately weaker than the bridge as a whole. Timber bridges are subject to decay and various accidents, and may develop structural weakness of parts, all of which require timely attention.

First-aid or emergency treatment involves, generally, the setting of trestle bents under the bridge. Where these cannot be permanent, many bridges may be strengthened by the addition of plank arches inside the truss, if side clearance permits. Vertical hanger rods that pass through needle-beams hung below the chords carry the bridge load to the arches.

Lattice bridges are sometimes struck by running ice during spring freshets, and the lattice plank projecting below the lower chord may be broken off, so as to weaken, seriously, the power of the lower chord to carry the floor-beam load. Hanging the lower to the second chord by simple rods or yokes is the usual remedy.

If bridges tend to get out of line, as some do, they are guyed with wire ropes or rods to well-fixed bolts or other permanent anchorage on the banks of the stream. Decayed or broken parts may sometimes be renewed in kind. A common fault in all types is insufficient strength in the floor system. This calls for additional floor-beams or stringers, or both.

Lower chord-stick connections in all types are often deplorably weak in panels near the middle of the span. Fig. 25 at *D* shows a defect exceedingly common. The pull of the single clamp tends to throw the end of the stick outward, which greatly aggravates the tendency of the head to split at the notch. The remedy for this is an additional clamp cogged in and bolted on the outside; or, if split, by various types of iron couplings shown in Fig. 28, or in Fig. 16, for use in a 4-in. plank. The couplings can be extended to strengthen the central section for a length of, say, 40 ft., in lattice bridges, using loop ends at the gibs and right and left-hand threads for a turnbuckle at the center. Multiple rods can be used in this manner without trouble. Another favorite method is to attach an auxiliary stick to the side of the chord, connected with effective keys and large bolts or pipe drift-pins.

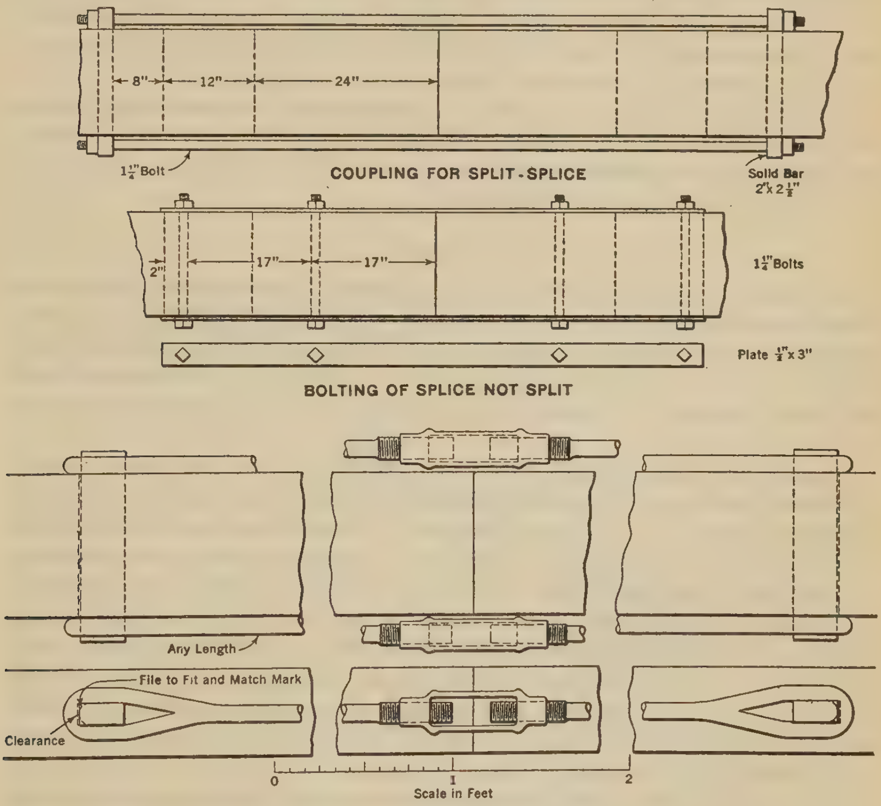
8"
12"
24"
1¼" Bolt
COUPLING FOR SPLIT-SPLICE
Solid Bar 2"x 2½"
2"
17"
17"
1¼" Bolts
Plate ½"x 3"
BOLTING OF SPLICE NOT SPLIT
Any Length
File to Fit and Match Mark
Clearance
0
1
2
Scale in Feet

FIG. 28.—REINFORCING COUPLINGS.

The tension rods in Howe and Pratt trusses are frequently found to be deficient in strength. Reinforcement is made by yoking a pair of rods to the truss with gib-plates of wood or iron across the chords, as near the panel points as possible. Reinforcement is more difficult in the Pratt than in the Howe trusses on account of fitting the gib-blocks to resist the horizontal pull of the inclined rods. The web rods in most of the older bridges were not upset, but facilities for upsetting are now such that its neglect in the long rods required for through railroad bridges is inexcusable.

The bridge builder cannot be too careful as to designing the bolted and spliced joints, in order to equate properly the stress and resistance where the parts come together. Manifestly, the tensile strength of coupling bolts and bars, the beam strength of gib-bars and bolts, the compressive resistance of cut-fiber surface and side grain of timber, and the tensile strength of the uncut section of the pieces united or joined, are all factors to be considered. Adjustment should be made so as to secure, with reasonable accuracy, equality of action and reaction throughout the combination.

Bridge Curiosities and Atrocities.—An interesting sidelight of the subject of timber bridges is the example, now and then, of the effect of ignorance and carelessness on essential details of construction. Three letters on this subject were published[39] in 1893.

SUMMARY

Reviewing in succession the advancing phases of this history, the writers have discussed the twisted-vine-cable bridge, in various adaptations to existing conditions, as the natural conception of primitive and early barbarian people. Then, in less barbarous and more semi-civilized times, the overhanging timber cantilever structure followed as an extension of the plain crib-work and log stringer.

Around the Mediterranean Sea, during the period of the colonizing and commercial activities of the Phœnicians and Greeks, there seemed to be little thought of bridges. Land commerce was borne by camels across the deserts, and by other pack animals in more settled regions. Wheeled vehicles were, for the most part, war chariots. The then abundant forests provided the timber for the ships sent forth from Tyre and Sidon. Ezekiel, the seer, wrote that the ship planks were made from fir trees, the masts from cedars of Lebanon, and the long oars from the oaks of Bashan.[40] A greater requisition on the forests was made later when the Greeks, Carthaginians, Persians, and Romans, in their long-prowed biremes and tiremes, fought the historic naval battles for the mastery of the inland sea.

It is conceded now that the conception of the arch as an architectural feature in masonry structures originated in the Near, or Farther, East. Quite recently archæologists have uncovered a tomb in Ur of the Chaldees, entered under a true arch of burnt brick. This is said to be 5 000 years old. In Europe, the extensive use of timber for bridges doubtless began when Rome first started to develop its expanding system of military and post roads. Cæsar's Rhine Bridge is a fine type of emergency bridge which would be creditable to a modern engineer. That the early Roman timber bridges embodied the preference for the semi-circular arch appears from the description of Trajan's Danube Bridge. After the long lapse of the Dark Ages, when the Renaissance brought new interest in public works, the flat arch became the standard type. It was expressed in open framing and bracing, such as proposed by Palladio; or solid timber arches, all usually with nearly level floors and side parapets, with no protection from the weather. The ponderous framing of some Swiss bridges like that at Schaffhausen, with their awkward lengthy bracing, found little acceptance. The low arch types, usually examples of good architecture as well as good engineering, prevailed during the Seventeenth and Eighteenth Centuries, until the later European timber bridges were displaced, during the Eighteenth and Nineteenth Centuries, by the splendid masonry bridges which stand as enduring monuments to distinguished engineers.

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[39] *Engineering News*, July 20, 1893, p. 60; September 14, 1893, p. 219; and November 9, 1893, p. 376.

[40] Ezekiel, XXVII, 5 and 6. About the Seventh Century B. C.

Since the aboriginal Americans had little use for products of the forest, the earlier American settlers along the coast had plenty of timber for pile and stringer bridges. Then Enoch Hale made courageous utilization of the magnificent white pine timber at his disposal for bridging the Connecticut River at the Bellows Falls gorge. Following him, the skillful and resourceful Timothy Palmer, and the ingenious and conscientious Lewis Wernwag built their great flat arches of wood somewhat stiffened by truss framework on top. Their insistence on careful protection from the weather inaugurated the distinctly American type of covered "through" bridge. In the first decade of the Nineteenth Centry, Theodore Burr brought out his idea of combining arches springing from the face of the masonry below the bridge seat, with a real truss, the lower chords of which carried the floor. This type of truss perhaps inadvertently on the part of Mr. Burr inaugurated a new era in bridge building, rendering it possible to trace the actions of loads from the point of their application to the supporting masonry.

At the end of the second decade Town brought out the lattice type; and, at the end of the third decade—in 1830—Colonel Long patented his design. These three types, Burr's, Town's, and Long's, are the only important ones, extensively used, that were practically constructed wholly of wood. From 1830, the building of railroads gave a great impetus to the building of bridges; and, in 1840, the Howe type with iron-rod vertical web members, became a favorite; and, in the next decade, the Pratt type, with iron-rod diagonals and timber verticals, soon led to iron lower chords; and then followed the so-called combination bridge interspersed with the early all-iron structures.

It is interesting to note the growth of the use of iron in bridge building. The building of clipper ships along the New England Coast up to the middle of the Nineteenth Century developed there the art of framing heavy timbers, the fastenings of which were made chiefly by treenails ("trunnels"). The iron used was fashioned by the smiths in the shipyards. Timothy Palmer, trained in those yards, used some slotted iron bolts, tightened by keys, as can be detected in Fig. 11. Use of threaded bolts became general afterward. Town used treenails only, but later practice in his type of bridge prescribes bolts freely, and rods where supplementary arches are used. In 1840, Howe made iron-rod verticals the basis of his claim, while Pratt, in 1850, extended their function as diagonals, and also made the curved upper chord a feature. The combination type called for iron diagonals and lower chords with compression members of timber. As early as 1840 all-iron bridges were built with cast-iron compression and wrought-iron tension members. The ingenious and scientific Squire Whipple, Hon. M. Am. Soc. C. E., enters the field here, using this combination of cast and wrought iron successfully for many years. In 1859, Howard Carroll built the first all wrought-iron bridge for railroad use. It was a riveted lattice truss, and the contest between the riveted and pin-connected types was on. In the last decade of the Nineteenth Century steel took the place of iron as the best material for bridges; and now, ninety years after Howe's introduction of iron as an integral part of the truss, other alloys of steel are coming to replace carbon steel in the art.

ACKNOWLEDGMENTS

Special acknowledgment is due to John W. Storrs, M. Am. Soc. C. E., Consulting Engineer, Concord, N. H., for co-operation in research and for valuable information freely given. Others to whom the thanks of the writers are due are, Frank B. Rowell, Research Engineer, Boston and Maine Railroad Company; Lyman S. Hayes, Town Clerk, Rockingham, Vt.; and Mrs. Mary N. Bolles, Bellows Falls, Vt. Several other correspondents have contributed important data without which the history would have been less complete.

DISCUSSION

Henry B. Seaman,[41] M. Am. Soc. C. E. (by letter).—This valuable paper is not merely a history of wooden bridges; it is practically a treatise on the design of wooden bridges of the more recent types. The Howe truss will be in vogue in timber countries for many years to come. The recent tendency toward iron bridges is partly the result of heavier engine loading; but it is due more especially to the increased price of timber, which would have less effect in countries where the timber is ready at hand.

Fifty years ago there were still many Howe truss bridges on the Pennsylvania Railroad. Although they were really combination bridges of wood and iron, they were classed as wooden bridges, and if iron was used unnecessarily it was considered an indignity to the wooden truss. Cast-iron angle blocks and wrought-iron tension rods were a regular part of the construction. The chord splices were of oak and were compactly bolted in place. In proportioning the splice, the area of the split section to that in the end thrust bearing was in a fixed ratio. Long spans (180 ft.), which still remained in service, had arch ribs on each side of the truss, which extended down below the chord to the face of the abutment, on skew-backs.

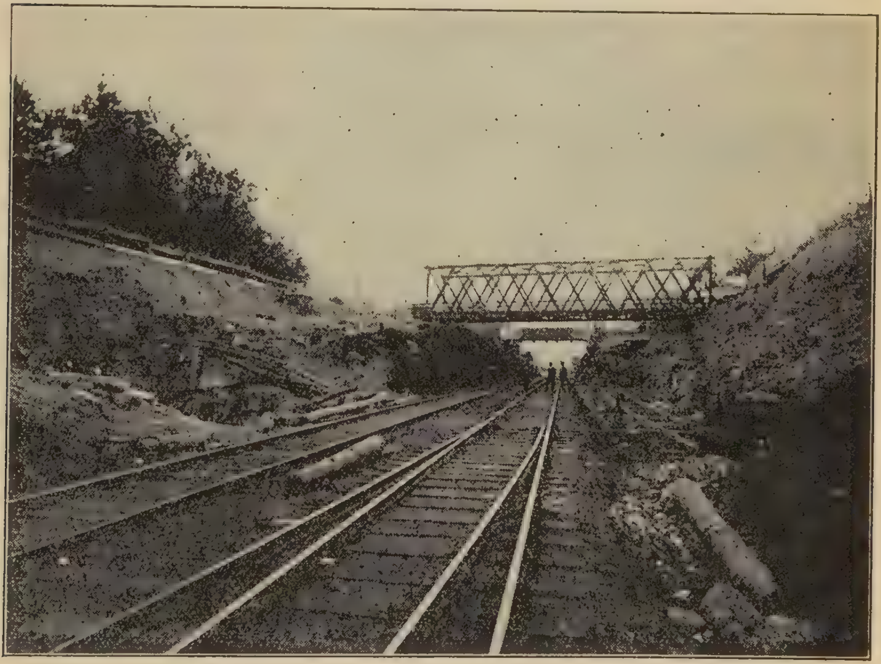

Fig. 29.—Timber Howe Truss, Mount Vernon, N. Y., 1893.

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[41] Cons. Engr., Brooklyn, N. Y.

On the Erie Railroad, about ten years later, there were many Howe truss bridges of shorter span, but without the arch rib. Since the Railroad Company was avoiding the appointment of a Receiver, the bridges were used to their ultimate of safety and were badly "hog-backed" before removal. They were tested at the critical joints by the use of a small auger to ascertain the condition of the interior wood, and the holes thus made were then filled tightly with plugs. It was the saying that "a wooden bridge never falls down," which was, of course, a dangerous doctrine; but it received apparent corroboration when the Market Street Bridge, in Philadelphia, Pa., was removed, and was found to have been literally punk.

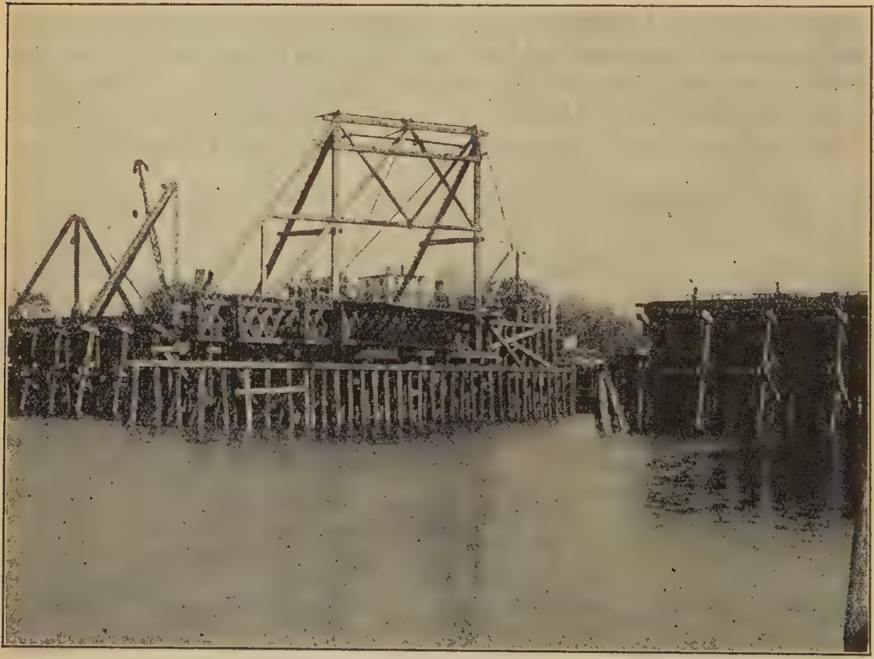

FIG. 30.—TYPICAL JACK-KNIFE DRAWBRIDGE, WESTPORT, CONN., 1905

About 1893, the writer had occasion to design and construct the overhead Howe truss bridge shown in Fig. 29, for a temporary crossing of the New York, New Haven and Hartford Railroad, at Mount Vernon, N. Y., and found no difficulty in obtaining trained carpenters to execute the work. It was of simple design, with oak angle blocks, a complete upper lateral system, and knee-braces only at the portals.

More could be written on the subject of the jack-knife draw-bridge. This is still a useful type for temporary purposes. While re-constructing the main-line bridges of the New Haven Railroad at Cos Cob and at Westport, Conn., in 1905, two such bridges, double-tracked, were constructed by the writer, and served well for main-line service during the reconstruction. One of them is shown in Fig. 30. These jack-knife draw-spans were part of the temporary by-pass trestles. For satisfactory operation of the draw-span, it is important that the gallows-frame be inflexible, and that the pile bents from which the trusses swing, be rigidly braced against movement.

JASPER O. DRAFFIN,[42] M. AM. SOC. C. E. (by letter).—Timber bridges of the type discussed in this paper are fast disappearing, particularly in Vermont where a large number have been replaced within the last ten years and especially since the flood of 1927. The writer is particularly interested in one of the bridges described, the Enoch Hale Bridge, at Bellows Falls, Vt., or the first bridge over the Connecticut River. The latest successor to this primitive bridge is a two-span, reinforced concrete arch built in 1930.

That this early bridge was considered important is shown by the account mentioned by the authors,[43] which reports that Enoch Hale "has just built" a two-span bridge, across the Connecticut River at the "Great Falls," 360 ft. long and 60 ft. above high water. It was believed to be the most elegant, the strongest, and one of the most useful bridges in America. The item went on record that while the work of building it was dangerous, the only casualties were the death of a young man who fell on the rocks in the river, and a slight injury to Colonel Hale.

The main route between Boston, Mass., and Montreal, Que., Canada, passed over a turnpike road through Keene, N. H., crossed the Connecticut River at Bellows Falls, and continued across the Green Mountains to Rutland and Burlington, Vt., and thence to Montreal by Lake Champlain. Consequently, it was an important artery of travel and transportation at that time and the Hale Bridge was a vital link in it. The reason for the selection of this place for the construction of the bridge was that the river is narrow (300 to 400 ft. wide), passing through a rocky gorge with a ledge in the middle, on which a center pier could be placed. A more favorable site for a bridge can scarcely be imagined; but not only was the location desirable from an engineering point of view, it had a considerable reputation for scenic beauty.

A question has been raised concerning the replacement of the original Hale Bridge. This arose mainly because Timothy Dwight stated[44] that, "in 1803 [the year of his first visit to Bellows Falls] the bridge erected by Colonel Hale * * * had been taken down and a new one built." He does not say that the replacement took place in 1803, but merely that it had been done by that time. In an earlier paragraph, he states that the original bridge was standing in 1797. Therefore, if his account is correct, a new bridge was built between 1797 and 1803 to replace the one built by Colonel Hale. Some doubt of the correctness of the Dwight account has been expressed, partly because there did not seem to be any local tradition of the replacement. However, there is a record that the owner of the bridge, a Mr. Guier (or Geyer), was trying to raise money by mortgaging his property at about the time of the supposed second building of the bridge. The bridge built by Colonel Hale has been described as a combination of arch and truss and neither the first bridge (see Fig. 8), nor the second one was of this type; but the description of the second bridge by Timothy Dwight would lead one to conclude that it was of the arch type, since he described it as "an obtuse arch." Taking into account all the uncertainties and the short time between the building of the first bridge and its replacement or rebuilding, it seems reasonable to conclude that the original bridge was not replaced in its entirety by a new bridge, but that it was merely closed to traffic and extensively repaired between 1797 and 1803. In the beginning of his search for information, the writer wished to believe that there was an entire replacement of the bridge, but the evidence obtained thus far does not warrant such a conclusion.

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[42] Assoc. Prof., Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

[43] *The Massachusetts Spy,* February 10, 1785.

[44] "Travels in New England and New York," 1803.

The reconstructed bridge lasted until 1840 when it was replaced by what was known locally as the "Tucker Toll Bridge," shown in Fig. 18, and which was of the covered lattice timber type with sidewalks on the outside of each truss. It was placed 15 ft. higher than the Hale Bridge which was used during the construction of the new one, after which the old bridge was cut away and allowed to fall into the river.

E. K. Morse,[45] M. Am. Soc. C. E. (by letter).—It is refreshing to read something that is not buried in a mass of intricate calculations that are known only to the author. Several facts have not been mentioned in this paper, namely, that many of the old wooden bridges, especially those constructed by Burr, were built of sized white pine, free of knots, adzed into shape, and then hand-surfaced on four sides. This was especially true in the Burr Arch Bridge across the Allegheny River at Ninth Street, in Pittsburgh, Pa. (which the writer contracted to remove in 1890-91), and also in a bridge of the same design built by Burr at Saltsburg, Pa., across the Kiskiminetas River.

In inspecting the bridge at Saltsburg some years ago, the writer found the name of one of the carpenters who evidently had planed the beautifully shaped and finished members of the arches. The man had engraved his name in the top of one of the arches which was just as bright and as clear as the year it was erected. All the timbers going into these bridges were seasoned from two to four years before being placed in position, which accounts for the accurate work and the condition of the bridges after years of heavy travel. On the Ninth Street Bridge there was not an open joint on any of the spans, averaging from 178 to 210 ft, center to center of skew-backs. The white oak flooring, like the white pine, was seasoned in the old-fashioned way and surfaced on all four sides. Teams and wagons passing over the floor moved so smoothly that the edges were not frayed. In practically all the bridges that were erected by Burr, the floors were originally laid dust-proof and because they were not exposed to the elements, they remained intact for many years.

In taking down the old Ninth Street Bridge, heavy vehicle and foot travel had to be maintained and, in order to facilitate erection, the roof was removed, the trusses were cut to the line of the arch, and $1\frac{1}{2}$-in. bolts were placed through the edge, and the verticals supporting the floor system clamped tightly to the arches wherever they showed signs of weakness. In order to prevent excessive vibration, a two-post tower with sway-bracing was erected at the center of the arches. There was considerable vibration in the arches when they were cut loose from the floor system and the tower was removed, but when 1½-in. manila rope was connected to the top of each arch and it was attempted to pull them over and dump them into the river, they yielded readily for about 3 or 4 ft and then resisted even the 5 tons of horizontal pull that was put on them, and eventually had to be removed panel by panel.

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[45] Engr. Member, Water and Power Resources Board of Pennsylvania, Pittsburgh, Pa.

It was found that the members in all the arches had completely lost their fiber to the extent that it was almost impossible to cut one of these timber arches with an axe. It was as difficult to lift a chip from one of these white pine timbers as it is with a red gum or green sycamore. The timbers were bright. During the erection of the new bridge, they supported a load three or four times as great as that for which they were designed and, in no case, did any member of the arches show any sign of weakness before, during, or after being taken down. A match factory in Chicago, Ill., had bought the timber in the white pine arches, and needless to say the contract was cancelled when it was discovered that white pine which is so easy to chop and so beautiful to work, had become as tough as gum.

JOHN W. STORRS,[46] M. AM. SOC. C. E. (by letter).—This paper is not only interesting and informative, but instructive, and must have required much thought, research, and investigation.

Old wooden bridges have recently been the subject of a number of brochures. Many people are showing an interest in these structures. As an exemplification, there was an unique service at the site of an 82-year old bridge in the Town of Boscawen, N. H., on the afternoon of December 3, 1932. This is a covered bridge of the Burr type with arches, and it spans the Contoocook River. It has been in constant use since 1850. Repairs have been completed recently, and it will do service for a long term of years to come.

The bridge at Woodsville (in the Town of Haverhill, N. H.) built in 1828, and the one in Bath Village built in 1832, are still in service and apparently taking care of the traffic in a satisfactory manner.

The writer would be one of the last to cast any reflections, to find fault with or to criticize the earlier American builders of bridges. They were limited as to means and material; had little to go by except trial and experience. Some of the so-called hybrid or mongrel bridges did, or have done, extraordinary and remarkable service, and the constructors and bridge carpenters of those days "did the best they could; angels could do no more."

C. J. HOGUE,[47] M. AM. SOC. C. E. (by letter).—This paper is one of great historical and informative interest and value, at least to one connected with the timber industry, and the writer hopes that it may be to others, as well.

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[46] Cons. Engr., Concord, N. H.

[47] Engr., West Coast Lumbermen's Assoc., Inc., Seattle, Wash.

It brings the history and practice in wood bridges to date, and the writer believes that the present is the eve of a fourth era in wood bridge design and construction, which will be of even greater interest and importance, from both engineering and architectural viewpoints, than the three recorded in the paper.

Temporarily, bridge design and construction in wood have been somewhat eclipsed by other materials. These materials have been new; wood construction has been old. They have been replete with new technical studies and developments; wood technology, old as it is, has lacked definite information on some points.

To-day, as not for many years, the bridge designer is faced with the necessity for the utmost economy and, probably as never before, with the question of the length of useful life and early obsolescence. A timber supply is amply available, recent research has established authoritative information on most points in question, new methods of framing have been developed, and treated timber offers material of economic permanence, ease of remodeling, low cost of removal, and salvage value on removal.

During recent years, the Forest Products Laboratory, of the U. S. Forest Service, at Madison, Wis., has published the results of technical studies which have taken the mystery and variance from the calculation of the bearing strength of wood posts, of bearing on wood at various angles to the grain, of the bearing strength of bolts on wood, of the calculation of shear in wood beams and stringers, and of the basic principles of calculating the strength of laminated wood beams and posts. The ring dowel connector, developed in Germany during recent years, opens a vast new field of economy and flexibility in wood framing.

It has been shown by experience that the durable life of treated timber may be assumed safely at forty years or more; it seems to be the consensus of opinion of bridge engineers that the useful life of a bridge in these times of rapid change may not be assumed safely at more than twenty to twenty-five years, often even less, due to early change in location.

The public has been taught to expect beauty as well as utility in its bridge structures. During 1932 a number of bridges of appealing architectural character, with little increase in cost, have been built in Oregon.[48] Such framing connectors as the ring dowel open the field for wood bridge construction to long-span arches and arched trusses of imposing size and appearance. The writer believes that a new chapter in wood bridge construction is about to be written.

William A. Oliver,[49] Assoc. M. Am. Soc. C. E. (by letter).—The reading of this paper has been a pleasure and an inspiration. The education of an engineer is not complete until he has become acquainted with the work of the giants who have gone before him. It is through such efforts as the one under discussion that this part of an engineering training is obtained.

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[48] *Civil Engineering*, September, 1932, Vol. 2, No. 9, p. 549.

[49] Urbana, Ill.

In their "Synopsis," the authors write, "but it will appear that, where timber is plentiful * * * wooden bridges still have 'a reason to be.'" Later, they state (see "Stringer Bridges"), "although it can be said that the day of wooden bridges has passed, it is a fact that many hundreds of miles of wooden trestles, stringer, and truss bridges are still in service in the United States." It is not entirely clear just what the authors' conclusion is regarding the present adaptability and economy of timber bridges. The writer would like to discuss the paper briefly from this point of view.

One of the important objections to timber bridges and structures generally has been the so-called "lack of permanency." The writer does not believe that the facts justify this idea. By proper maintenance a timber bridge may be just as permanent as a bridge built of some other material. For centuries, and by consistent and precise maintenance, the Japanese have made their timber bridges perpetual. Numerous examples of timber bridges several decades old can be found in the United States, and many of these bridges have had only ordinary care during these years of constant use. Certainly fifty years is a long life for any ordinary bridge of any material in the United States. Some examples of present-day wooden bridges are briefly discussed in the following paragraphs.

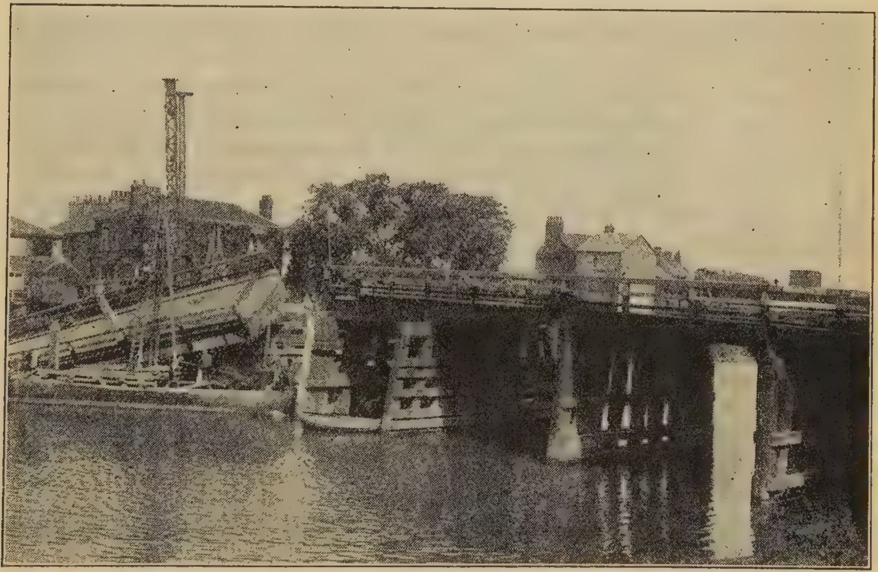

FIG. 31.—SELBY BRIDGE OVER THE RIVER OUSE IN YORKSHIRE, ENGLAND.

According to Mr. E. Jervoise,[50] Selby Bridge over the River Ouse, in Yorkshire, is probably the oldest timber bridge in England. It was built in 1790 and is still in use (1933). The bridge has a swing section which, in Fig. 31, is shown thrown from its pivot. Mr. Jervoise states that through this mishap the bridge was out of use for some time.

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[50] Ancient Bridges of England Series, Architectural Press, Westminster, S. W., London, England.

Of course, modern examples of wooden bridges are plentiful on the West Coast of the United States, but, in 1932, the State of Ohio found it expedient to build ninety-three timber highway bridges. Most of these structures were of a simple trestle type, built of timber produced in the State. All material was impregnated with preservative, all framing being done in advance of treatment. It is interesting to know that a mid-Western State had timber in this quantity available for use, and that one of the more progressive States did find it economical to adopt such an elaborate program of timber bridge construction.

One of the earliest of materials of construction, timber has been developed for structural purposes by "rule-of-thumb" methods. Due to the inertia that these "rule-of-thumb" methods obtained through the years, it has been difficult to make the ordinary timber user appreciate the fact that greater economy as well as strength could be obtained through a more scientific use of the material. However, through the efforts of the various lumber manufacturers' associations and the work of the materials testing laboratories, this difficulty is gradually being overcome and the general trade is evincing more and more interest in researches in the use of timber. Many examples of present-day research could be given as examples of what is being done along these lines. As far as the testing laboratory is concerned timber is almost an ideal material, conforming beautifully with the accepted theories of mechanics.

Referring to the work of Messrs. Brangwen and Sparrow,[14] the authors state in their discussion of Roman bridges, that very little is known of the construction of Pons Sublicius, "hallowed by traditions." A longitudinal elevation and a cross-section of this bridge has been published[51] by Mr. H. G. Tyrell. The writer wonders whether the authors question the authenticity of this illustration.

Philip G. Laurson,[52] M. Am. Soc. C. E. (by letter).—The authors have contributed a paper of great value to civil engineers who are interested in bridges. Much of the information is not available elsewhere.

It is natural and proper to emphasize the American developments as the authors have done. It is true, however, that wooden bridges were used in Europe a century ago to a greater extent than is commonly realized.

The *Minutes of Proceedings* of the Institution of Civil Engineers of Great Britain, and other European engineering publications, contain numerous descriptions of wooden railway and highway bridges. In 1846, John Rennie, in his Presidential Address, reviewed the developments in all branches of Civil Engineering, including wooden bridges. He reported at that time[53] that the trussed system had been applied with considerable success in some well-constructed bridges across the Tyne River and in several other places. The system of combining small curved pieces of timber connected in the form of an arch, adapted for large spans, was first introduced by C. F. Wiebeking in 1826, according to Rennie, when a bridge of 100-ft span was constructed on the

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[51] "History of Bridge Engineering," p. 23.

[52] Assoc. Prof. of Eng. Mechanics, Sheffield Scientific School, Yale Univ., New Haven, Conn.

[53] *Minutes of Proceedings*, Inst. C. E., Vol. V (1846), p. 33.

Ancholme with complete success. He reported also that the lattice bridge had recently been introduced on the Birmingham and Gloucester Railway and on the Dublin and Drogheda Railway and that it had been found economical and simple in construction. A number of European types of wooden bridges, with details, was described by Benjamin Green in the same year[54] in a paper entitled "Arched Timber Viaducts."

Isambard Kingdom Brunel designed many timber railway bridges in a great variety of types.[55] Brunel was particularly concerned with the prevention of decay and, as early as 1835, was in communication with Faraday as to the best method of testing the extent to which Kyanizing solution penetrated into wood. It is reported that Brunel made a careful study of all the different methods of preserving timber, and used the more successful of them on a very considerable scale.

Many of the European engineers of these early times were well-educated men of high technical skill. Carl Friedrich von Wiebeking (1762-1842), mentioned by Rennie, was Director-General of Roads and Bridges in Bavaria, and was the author of several treatises on hydraulics, mechanics, and structures. He worked out proportions for his laminated wood arches for spans as long as 600 ft. Bridges of this type were built in England and on the Continent in considerable numbers. The authors' Fig. 6 is of this type, but in most of the bridges the bent planks were thinner — 3 in. being a common thickness. As many as sixteen or twenty thicknesses of bent planks were bolted together to form the arch rib.

Another good source of information concerning European timber bridges is the set of publications on the "Theory, Practice, and Architecture of Bridges," edited and published by John Weale about 1843; it contains two volumes of fine plates. The early volumes of *L'Illustration* also contain pictures of wooden bridges in the days when railroads were still objects of curiosity.

It is true, of course, that because the use of timber was less advantageous in much of Western Europe, than it was in the United States, bridges of other materials have always predominated.

The practice of assembling trusses on land before erection must have been more common than the authors imply. David Stevenson, a Scottish civil engineer, made an extensive tour of the Eastern United States and Canada, about 1837, for the sole purpose of inspecting engineering works.[56] He reported seeing many Town truss bridges on the Philadelphia and Reading, and on other, railroads.

He also noticed the Long bridge in many parts of the country, the best specimens being on some of the railways in the neighborhood of Boston, Mass. His account states, furthermore, that:

> "The timbers of which Town's and Long's bridges are composed, are fitted together on the ground previous to their erection on the piers. They are again taken asunder, and each beam is put up separately in the place which it is to occupy, by means of a scaffolding or centering of timber."

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[54] *Minutes of Proceedings*, Inst. C. E., Vol. V (1846), p. 219.
[55] "Life of I. K. Brunel, Civil Engineer," by Brunel, Lond., 1870.
[56] "Sketch of the Civil Engineering of North America," Lond., 1838.

Richard S. Kirby,[57] M. Am. Soc. C. E. (by letter).—The authors of this paper have covered in a thorough and interesting fashion a field too long neglected. The writer was especially pleased with their emphasis on personalities of the period. In this connection he ventures to add a word concerning the McCalls Ferry Bridge over the Susquehanna River, the work of Theodore Burr, Connecticut-born engineer and contractor, of Harrisburg, Pa. This bridge was one of Burr's great achievements, but little is heard of it because of the fact that it was carried down by the ice in 1817 after only two years' service. The writer believes that it had the longest span of any wooden bridge ever attempted in America, and that only that of the Schaffhausen Bridge had exceeded it.

In a long letter written by him shortly after he finished this bridge, to his friend, Reuben Field, bridge builder of Waterford, N. Y., Burr rehearsed graphically his thrilling experiences in erecting the bridge during the winter of 1814-15. The arches were fabricated in halves and raised at first on bents of varying heights supported by floats ranged along and parallel to the shore nearly $\frac{1}{4}$ mile below the site. As the ice increased in thickness, Burr decided to use it instead of the floats. He finally slid the entire structure up stream on rollers. Man-power was abundant in those days, in that locality at least, for Burr commends the inhabitants for their willing help, which was perhaps gratuitous.[58]

Wells N. Thompson,[59] Assoc. M. Am. Soc. C. E. (by letter).—In the early stages of the development of wooden bridges, the crib or corbel type was one of the most interesting. Examples of this type of bridge are fast disappearing as they are replaced by more modern structures, but there are a few remaining. Fig. 32 shows a very interesting one in the Far East.

This old bridge is well known throughout Japan, and it is shown or mentioned in many native wood-block prints, poems, and songs. It is located in Central Japan, Yamanashi Province, and supplies its own name to the nearest town, "Saruhashi" or "Monkey Bridge." It is fifty miles west of Tokyo and spans a mountain stream at a point where it is approximately 80 ft wide between abutments. Age is evident in the entire structure, but repairs and replacements have been made as required so that it continues in good condition and in general use for the traffic in that district, which consists largely of pedestrians, jinrikishas, and ox-carts. The roadway is about 12 or 14 ft wide.

The Japanese excel in timber construction, and are very careful about protecting their structures against rot and decay. In this bridge, as in all other structures where framing is exposed to the weather, they have protected each member in the supporting cribs with an individual roof or watershed. Such care has preserved many of their very old and notable structures.

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[57] Assoc. Prof., Eng. Drawing, Sheffield Scientific School, Yale Univ., New Haven, Conn.

[58] The letter appeared in *Niles Weekly Register* (Baltimore, Md.), November 18, 1815. It is copied in the Lancaster Historical Papers, Vol. XI, p. 133.

[59] Danbury, Conn.

The abutments are field stone masonry, built against the rock walls, and have probably been added recently, in rebuilding or reinforcing the old stonework, perhaps after damage by earthquake.

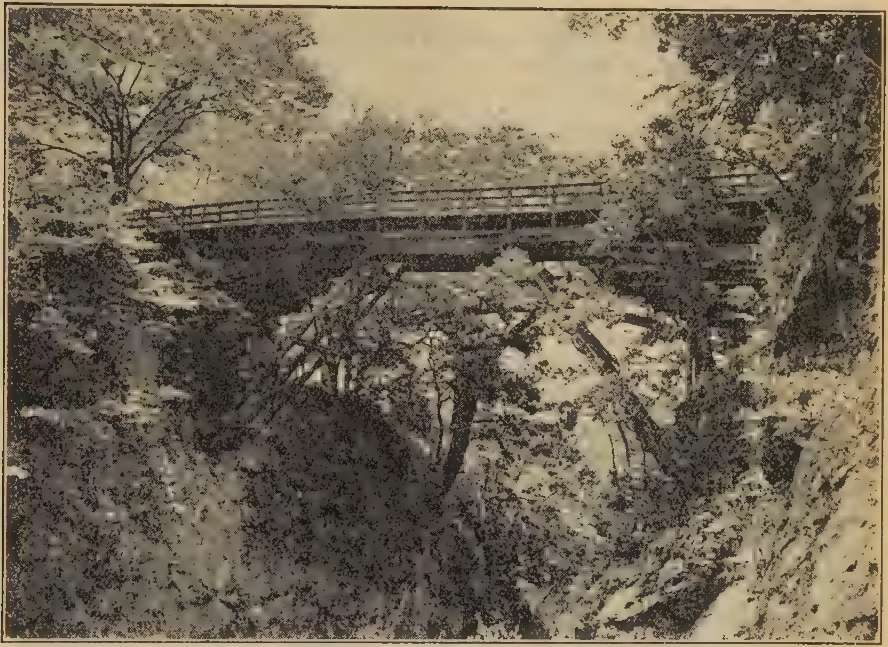

FIG. 32.—OLD CORBEL TYPE OF BRIDGE, YAMANASHI PROVINCE, JAPAN.

BENJAMIN WILDER GUPPY,[60] M. AM. SOC. C. E. (by letter).—In connection with Fig. 28 of the paper, the authors point out the important warning that the bridge builder cannot be too careful in designing the bolted and spliced joints of a timber bridge. A few examples should serve to emphasize this importance.

A draftsman who has had experience only in detailing metal structures has considerable difficulty when he first attempts to design a timber structure. In such a case the writer has told the man that if he designs a steel bridge while standing on his feet, he should stand on his head while designing a timber structure. In other words, the processes are reversed. In steel designing, the sections of the main members are determined first and then the details designed to transmit the stress properly. In straight timber work it is the design of the connection that generally determines the dimensions of the main members. In metal work the unit stresses to be considered are those of tension, compression, shear, and bearing. In timber the unit stresses to be accounted for are tension, compression, bearing on end grain, bearing across grain, shear parallel to fiber, and shear normal to fiber on planes parallel to and normal to direction of fiber.

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[60] Engr. of Structures, B. & M. R. R., Boston, Mass.

The simplest case is a post in a trestle bent. In a steel structure the area of the post is determined by dividing the stress it is to carry by the allowable unit stress determined by some approved column formula. The dimensions of the timber post are determined by the stress in the post divided by the allowable stress in bearing across the grain on the cap or sill. A moment of reflection will show that it will be a very long slender post to require an allowable unit compressive stress as low as the unit bearing stress on side grain of timber.

The next step in advance is a king-post truss and to illustrate his statement the writer presents a design using timber entirely, use of iron being restricted merely to small bolts to hold the various pieces together (see Fig. 33). The king-post is in tension. To determine its proper dimensions both upper and lower connections must be considered. At the lower connection the size of the post is determined by the net area required in tension, plus the area required to transmit the stress by bearing across the grain. At the upper connection the area of the post is determined by the net area required to carry the tension, plus the area required to transmit the stress by bearing on the end grain. The larger of these areas governs. At the upper joint the strut must be framed both to give sufficient area for the vertical component in the strut at the unit stress for bearing on the end fiber and the horizontal component in the strut at the unit stress for bearing across the grain. The distance from the daps to the end of the king-post must be sufficient to prevent the timber from shearing beyond the daps. At the bottom of the strut the connection is reversed from that at the upper end as the horizontal component must be carried into the bottom chord through the end bearing and the vertical component through the side bearing. The bottom chord must extend far enough beyond the dap to give sufficient strength in the shear along the grain. The size of the bottom chord at the end joint is determined by the depth of dap necessary to transmit horizontal stress from the strut and

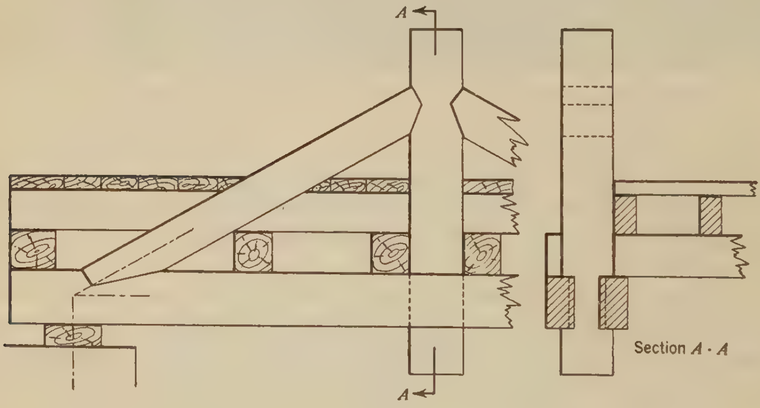

FIG. 33.

the depth of stick required below the dap to carry the tension in the bottom chord plus any bending that may be induced by eccentric connections or methods of loading.

When it is necessary to splice a tension member the design of the splice is necessary before sizes of timbers can be determined. This is not submitted as an example of good modern practice, but to illustrate principles of design in timber and to show what will have to be done if metal for details is not available. The use of metal angle blocks to distribute stress at connections will permit of detailing compression members in accordance with an approved column formula and the use of rods for tension members and joint bolts will greatly simplify detailing connections. Figs. 16 and 28 in the paper by Messrs. Fletcher and Snow show how efficiency of tension splices can be increased by a free use of metal.

IVAN E. HOUK,[61] M. AM. SOC. C. E. (by letter).—The authors have contributed an extremely interesting and valuable account of the history, design, construction, and life of wooden bridges.

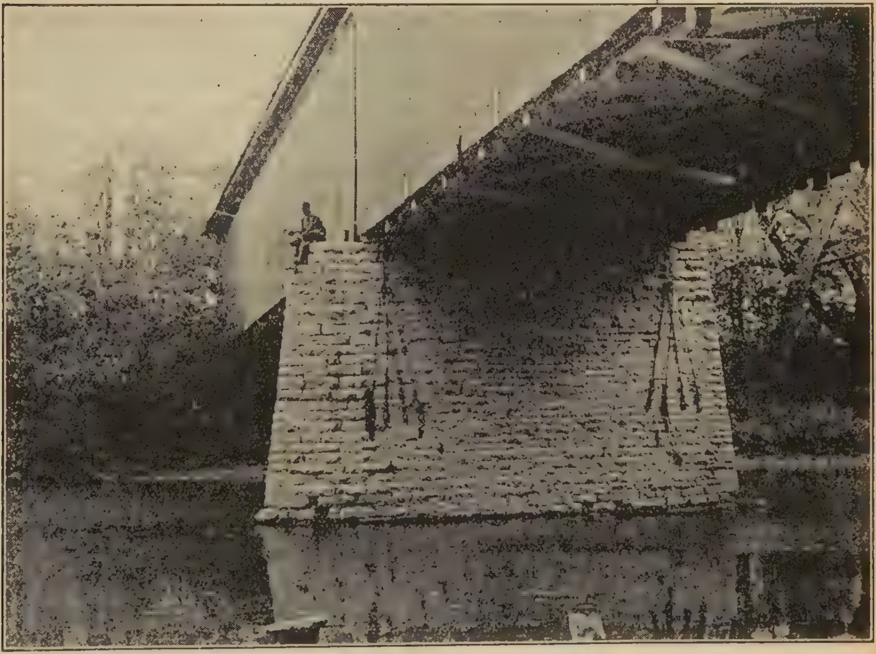

FIG. 34.—VIEW OF TIMBER TRUSS COVERED HIGHWAY BRIDGE OVER THE STILLWATER RIVER IN SOUTHWESTERN OHIO.

In connection with their discussion of retrieving weak and injured bridges, an account of the rather novel repairs shown in Fig. 34 may be worth while. This bridge is one of the old timber truss, covered highway bridges, still so common in the Eastern United States. It spans the Stillwater River, of

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[61] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

Southwestern Ohio, about two miles above the Englewood Dam. It withstood, safely, the disastrous flood of March, 1913, as well as all the previous floods that had occurred after it was built. When it was built, or whether it is still in existence, is not known. However, it must have been about fifty years old at the time the Englewood Dam was completed. The Englewood Dam is one of the five large flood-prevention dams built by The Miami Conservancy District about 1920 or 1921.

Prior to the completion of the flood-prevention works the bridge was located on the National Road and carried all through traffic along U. S. Highway 40 in that section of the State. Since that time through traffic has been routed over the top of the dam. However, the bridge was still in good condition at the time the dam was built, and it was desired to keep it in operation after the through traffic was diverted. Consequently, it was necessary to anchor it in place, to keep it from floating away during flood periods when the back-water above the dam might rise 30 or 40 ft above the roadway surface. In other words, the construction of the dam below made it necessary to hold the bridge down as well as up.

Iron rods, 1¼ in. in diameter, with an eye at the bottom and threads at the top, were placed vertically between the timbers of each truss, a rod being placed on each side of the pier and at each abutment. The upper end of the rod passes through the heavy timber which constitutes the upper chord and is held in place by a large iron washer and nut. The eye at the lower end is tied to the foundation by four steel cables, attached to iron bolts set in the masonry about 1 ft apart and about 12 ft below the lower chord.

The old, timber truss, covered bridges in the Miami Valley went through the destructive flood of March, 1913, as well as, if not better than, those of more modern design. Of course, this was because the masonry foundations had been either founded on rock or carried down to a sufficient depth below the bed of the stream, and had then been built to a height well above the maximum previous high-water level before placing the superstructure.

William H. Baker, Jr.,[62] Assoc. M. Am. Soc. C. E. (by letter).—The various developments in the use of wood for the construction of bridges, is illustrated clearly in this paper. It covers the development from the most simple type, through all the stages of design, to the present standard. The writer believes that the development of the wooden bridge has been a great influence on the history of the world, both in military conquest and social growth through the medium of communication. The wooden bridge is still used extensively in the United States, despite the great progress in the use of steel and reinforced concrete in the last twenty years.

The wooden bridge is still used much more extensively than bridges of steel and reinforced concrete in Los Angeles County, California. There are approximately six hundred bridges of wood construction in use at the present time. They consist principally of A-frames up to 42-ft span, together with pony Howe trusses for longer spans. The reason for this extensive local use of the wooden bridge is due to the inadequate finances available to construct a more permanent type of crossing over the many dry washes in the county—most of them dry, except during flood stages—and also because as yet flood-control channels have not been established in many instances. Some of these wooden bridges over dry washes are as long as 800 to 1 000 ft, and to have built them of steel or concrete would have increased the cost many times that of the wooden structures; and after the flood-control channel had been established definitely, the channel would perhaps be much narrower than the original bridge span.

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[62] Cons. and Structural Engr., Long Beach, Calif.

Since about 1925 or 1926 it has been necessary to widen many existing highways in Los Angeles County. The early bridges had 20-ft roadways—inadequate for present traffic needs. Most of the bridges now constructed have a roadway width which will be sufficient to provide for present traffic demands and for future widening of the highway. It was necessary, therefore, that the County Road Department adopt a policy in which all future bridges would provide roadways of at least 30 ft. The wooden bridges, both the **A**-frame and the Howe truss type, were constructed using large Oregon pine floor-beam members. These floor-beams for a bridge without skew were 24 ft long for a 20-ft roadway, being made up of two 12 by 36-in. timbers bolted together. For skew bridges, the floor-beams were longer, as the angle of skew increased, making these beams from 24 ft to approximately 36 ft in length. It was very difficult to build wooden bridges for a 30-ft roadway utilizing wooden floor-beams, due to the length requirement of such a large wooden section. At the same time, the cross-sectional area required for the wooden floor-beams greatly reduced the head-room and opening for the waterway.

At the time that the Carnegie beam sections using flat flanges were introduced, the Bridge Department of Los Angeles County designed and developed a wooden truss bridge of the Howe type for 30-ft roadways, and for various angles of skew, utilizing the new Carnegie beam sections of 30, 36, and 42-in. depth for floor-beams replacing the old wooden type beams. A very simple method of constructing a proper connection for the hanger rods and extension brackets for the pony truss braces was designed and developed. This use of the Carnegie beam section as floor-beams, which has now (1933) been standard for five years, permitted the design and construction of wooden bridges of 30-ft roadway and a much greater angle of skew than was possible with wooden floor-beams. At a later date, about two years after steel floor-beams were utilized, the lower tension chord member was designed, with steel channels to replace the old, built-up, wooden type lower chord.

The principal difficulty experienced with bridges of the wooden type was the dry rot that developed in the end-posts and flooring. Several types of construction have been tried over a period of years to eliminate this, including the use of 2 by 6-in. timbers of Oregon creosoted pine, laid on edge and the utilization of diagonal planking overlaid with transverse planking at right angles and surfaced with asphalt or macadam.

It is impossible to state or give true results at the moment as to which type or method of wooden floor construction is superior. Later, due to the difficulty in maintaining the wooden type of floor construction, a reinforced concrete floor-slab system placed on timber stringers was tried as an experiment. Several bridges utilizing such construction have been in place for about four years, and to the present time (1933), are in good condition. However, it is problematical whether this method will prove to be entirely successful and will replace the old method of wooden floor construction. The bridges that the writer last described are not strictly wooden, but are a combination of wood, steel, and concrete.

The use of the wooden bridge in Los Angeles County will probably continue for some years to come, until flood-control channels are definitely established and until new means of financing permanent structures of concrete and steel are advanced. The use of the wooden bridge is very costly, due to the heavy maintenance charges involved. However, they will have to serve until something more economical from the standpoint of first cost is substituted. This paper is of unusual interest and is a great contribution to civil engineering history.

Oren Reed,[63] Assoc. M. Am. Soc. C. E. (by letter).—The authors are to be commended for their excellent history of the development of different types of wooden bridges, a class of engineering structure that is fast disappearing in Central and Eastern United States. Since the structures described were located in the Eastern States, the writer wishes to refer to a few wooden bridges of Indiana to show the wide range in the use of such bridges. Most of these old structures, which are still in use, are on little back roads, off the main highways, and in out-of-the-way places; but a few are on major routes, such as the multiple-span bridge on the present U. S. Route No. 50 over the East Fork of White River, near Medora, and the two-span bridge on State Road No. 52, at the edge of Rushville, over Flat Rock River.

Nearly all the early covered bridges in Indiana were of the Burr truss type. The Howe truss soon superseded the Burr truss, to a large extent, in the East, but was not commonly used in Indiana, and probably not at all before 1865. The National Road, the Michigan Road, and the Vincennes-New Albany Pike were the early important roads and most probably the ones on which the first permanent bridges were built. For most of their lengths, these have remained important roads, and only a few of the original bridges remain.

Mr. J. J. Daniels, of Rockville, and the Kennedy family, of Rushville, are largely responsible for the great number of excellent timber bridges in the State, many of which are still in use. Mr. Daniels built about fifty bridges, mostly in Parke and Vermillion Counties. He built a bridge over Sugar Creek, in Parke County, in 1861, which is still in use. This structure has a clear span of 200 ft and is the longest single-span covered bridge of record in Indiana. The Kennedy family built about sixty covered bridges in all, the first being at Dunlapsville, in Union County, in 1870, and the last near Fountain City, in 1918. Many of these bridges are still in use in Southeastern Indiana. The Burr truss bridge, at Rushville, built by the Kennedys in 1883, has two 125-ft spans with a 16-ft clear roadway, and is unusual in that it has an arcaded sidewalk on each side.

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[63] Constr. Engr., Emergency Conservation Work, Camp 2, Lincoln City, Ind.

Two of the oldest timber trusses, still in service, are the Ramp and Raccoon Creek Bridges formerly on State Road No. 43, near Greencastle, which were built in 1837 and 1838. These bridges were constructed of local material, except for a few hand-made iron bolts and brace-pins. Both have two 11-ft roadways carried by three trusses, one of which separates the two roadways. The Ramp Creek Bridge has a clear span of 91.5 ft and the Raccoon Creek Bridge, one of 120 ft.

The Ramp Creek Bridge was removed in 1932 and re-erected over Salt Creek, at the north entrance to Brown County State Park, by the Indiana State Highway Commission. The arch ring, top chords, rafters, and roof framing are of yellow poplar; the posts, lower chords, and floor are of oak; while the floor-beams, joists, and sub-flooring are of walnut. Members that had decayed to any extent, were replaced by new timber. Oak was substituted for the damaged walnut. In all, 10 to 15% of the members were replaced. The abutments, which were carried to rock, are of concrete, faced with sandstone masonry. This timber bridge will have ample strength to carry the park traffic and should stand for many years as a monument to the early bridge engineers and their ingenuity in the use of local materials. Raccoon Creek Bridge is also to be replaced by a modern bridge to care for the heavy through traffic on State Road No. 43. Another timber bridge, maintained by the Indiana Department of Conservation, spans Sugar Creek, in Turkey Run State Park, at the Narrows Crossing. It has a span of about 100 ft.

R. P. DAVIS[64] and L. L. JEMISON,[65] MEMBERS, AM. SOC. C. E. (by letter).—The authors of this paper merit the thanks of the Engineering Profession for their effort to record the history—from an engineering standpoint—of the fast-disappearing wooden bridge. Much of a general nature has been written on this subject in the last few years, most of which is, however, not from the engineer's point of view.

During the latter part of the Eighteenth Century, in building highways to connect the Ohio country with the Eastern seaboard, the stone arch was widely used, as exemplified by those on the National Highway between Hagerstown, Md., and Wheeling, W. Va. A little later the wooden bridge became the more popular, and from the beginning of the Nineteenth Century until after the Civil War wooden bridges were very common west of the Allegheny Mountains, the Burr type being almost exclusively used, without arches for the shorter spans. Nearly all these bridges were protected with roofs and side covering, with an open space at the top to permit proper ventilation.

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[64] Dean, Coll. of Eng., West Virginia Univ.; Cons. Bridge Engr., West Virginia State Road Comm., Morgantown, W. Va.

[65] Bridge Engr., West Virginia State Road Comm., South Charleston, W. Va.

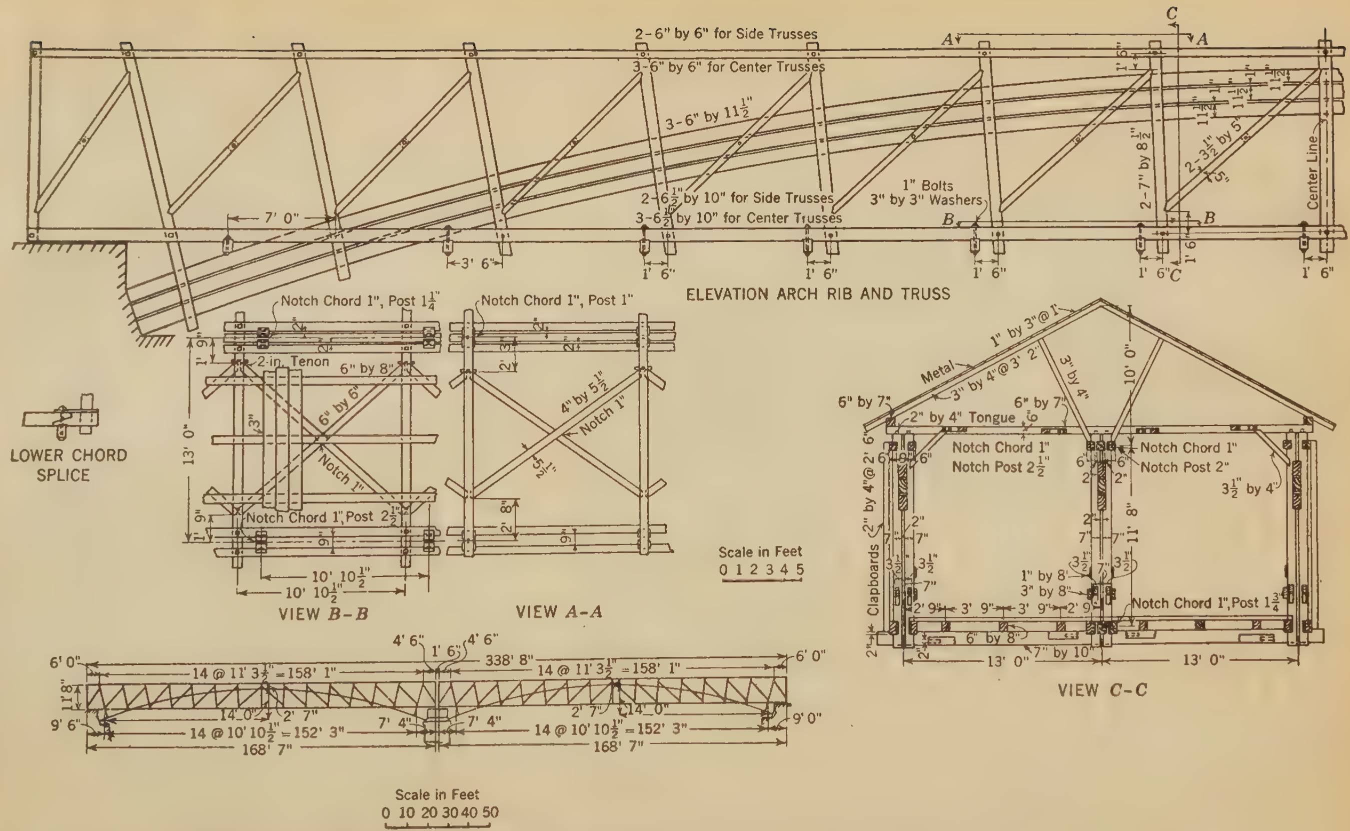
2-6" by 6" for Side Trusses
3-6" by 6" for Center Trusses
3-6" by 11½"
2-6½" by 10" for Side Trusses
3-6½" by 10" for Center Trusses
1" Bolts
3" by 3" Washers
2-7" by 8½"
2-3½" by 5"
5"
Center Line
7' 0"
3' 6"
1' 6"
1' 6"
1' 6"
1' 6"
1' 6"
A
A
B
B
C
C
ELEVATION ARCH RIB AND TRUSS
LOWER CHORD SPLICE
Notch Chord 1", Post 1¼"
2-in. Tenon
6" by 8"
6" by 6"
Notch 1"
Notch Chord 1", Post 2½"
13' 0"
10' 10½"
10' 10½"
VIEW B–B
Notch Chord 1", Post 1"
4" by 5½"
Notch 1"
5½"
VIEW A–A
Scale in Feet
0 1 2 3 4 5
Metal
1" by 3" @ 1'
3" by 4" @ 3' 2"
3" by 4"
10' 0"
6" by 7"
6" by 7"
2" by 4" Tongue
Notch Chord 1"
Notch Post 2½"
Notch Chord 1"
Notch Post 2"
3½" by 4"
2" by 4" @ 2' 6"
Clapboards
11' 8"
1" by 8"
3" by 8"
Notch Chord 1", Post 1¾"
2' 9"
3' 9"
3' 9"
2' 9"
6" by 8"
7" by 10"
13' 0"
13' 0"
VIEW C–C
6' 0"
4' 6"
1' 6"
4' 6"
338' 8"
6' 0"
14 @ 11' 3½" = 158' 1"
14 @ 11' 3½" = 158' 1"
11' 8"
14' 0"
2' 7"
2' 7"
14' 0"
9' 6"
7' 4"
7' 4"
9' 0"
14 @ 10' 10½" = 152' 3"
14 @ 10' 10½" = 152' 3"
168' 7"
168' 7"
Scale in Feet
0 10 20 30 40 50

FIG. 35.—BRIDGE OVER CHEAT RIVER, NORTHWESTERN TURNPIKE, PRESTON COUNTY, WEST VIRGINIA

Of particular historical value is the two-span combination arch and truss bridge over the Cheat River above Rowlesburg, W. Va. (see Fig. 35). This structure is on the old Northwestern Turnpike (later known as Federal Route 50), a highway built in 1832 under Col. Claudius Crozet, who had been an officer of artillery under Bonaparte in the Russian campaigns.

Completed in 1834 it has given excellent service for a period of more than 98 years. The material is white pine with all timbers hewn by hand; all bolts are of wrought iron with large heads. The bridge was built at a cost of $18 000, or $2.40 per sq ft of roadway, on the basis of a total length of 339 ft, and a clear roadway width of 22 ft. The arch ribs are composed of three 6 by 11½-in. timbers, placed vertically one above the other, an arrangement not as common as placing them side by side horizontally. However, by this arrangement (see Fig. 35), less notching is required, and the full section of the arch may be secured. The stringers, 6 by 8 in. in section and spaced 3 ft 9 in. apart, are larger than those customarily found in highway bridges of the early days. The 7 by 10-in. floor-beams are hung from the lower chords by two 1-in. bolts. Why the more common and simpler method of placing these beams above the chords was not used, is not apparent. However, this method of support gave complete satisfaction, because the original bolts are still (1933) in an excellent state of preservation.

It is interesting to note that the tension web members of the trusses are not placed vertically, as is more usual, but inclined in a direction approximately normal to the arch ribs. Loads from the floor-beams first come to the trusses and are transferred to the arch rib by notching the so-called verticals and by placing wedges between the three rows of timbers forming the arch rib. It will be observed that these trusses have only a single system of diagonals. As these diagonals are framed so that they take only compression, the question naturally arises as to why a double system was not used. The sticks composing the arch rib and the lower chords are three panels in length. The type of splice used for the lower chord would be considered now a very poor one for a tension connection.

During recent years a double floor has been used on this bridge, together with longitudinal runways. These were necessary on account of the wide spacing of the stringers. When heavy truck loading was introduced, considerable weakness developed in the floor-beams in resisting horizontal shear near the ends. This weakening was due to the notching of the floor-beams at their ends and also to the cutting of holes to permit framing the diagonals of the lower lateral system.

Most of the timbers composing the trusses and the arches are in an excellent state of preservation, except the arch ribs near the springing points where, due to failure to carry the siding down in the early years, considerable rotting developed. The bridge has also suffered under the action of some severe storms, as a consequence of which the trusses and arches are badly bowed out of line. The substructure is composed of stone masonry laid with close joints, without mortar. It is in good condition.

The bridge illustrated in Fig. 36 was on the famous James River and Kanawha Turnpike which was opened to traffic in 1800. The bridge was built in 1872 across the Greenbrier River, at Caldwell, W. Va. It had two spans, the arches being 228 ft long. Like the Cheat River Bridge it had a double roadway, the outside trusses being 25 ft, center to center. It was the second bridge to be built at the same location; the first one, constructed in 1807, was burned during the Civil War. The second bridge was built at a cost of $12 500, or $1.40 per sq ft of roadway, the superstructure resting on the original pier and abutments. When this masonry was torn down in 1932, the piers and abutments were found to be composed of stone masonry shells backed with small stones.

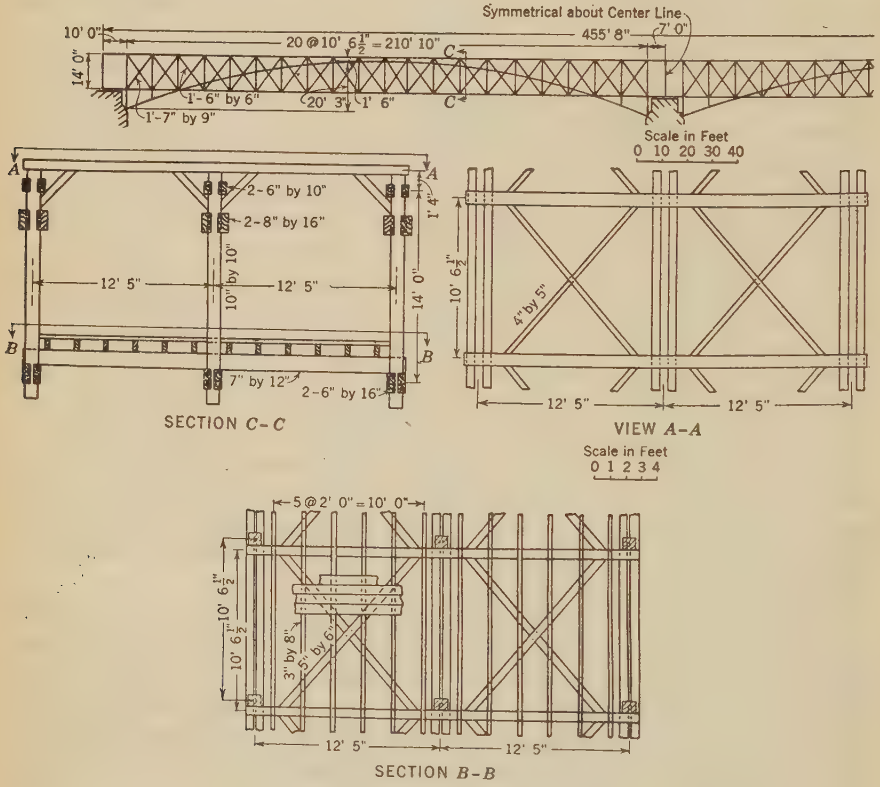
Symmetrical about Center Line
10' 0"
20 @ 10' 6½" = 210' 10"
455' 8"
7' 0"
14' 0"
C
1'-6" by 6"
1'-7" by 9"
20' 3"
1' 6"
C
Scale in Feet
0 10 20 30 40
A
A
2-6" by 10"
2-8" by 16"
1' 4"
12' 5"
10" by 10"
12' 5"
14' 0"
B
B
7" by 12"
2-6" by 16"
SECTION C-C
10' 6½"
4" by 5"
12' 5"
12' 5"
VIEW A-A
Scale in Feet
0 1 2 3 4
5 @ 2' 0" = 10' 0"
10' 6½"
10' 6½"
3" by 8"
5" by 6"
12' 5"
12' 5"
SECTION B-B

FIG. 36.—BRIDGE OVER GREENBRIER RIVER, AT CALDWELL, W. VA.

The floor system has 3 by 8-in. stringers spaced on 2-ft centers, resting on 7 by 12-in. floor-beams, which, in turn, rest on the lower chords of the trusses. As the trusses and arches resemble very closely those to be described hereafter for the Philippi Bridge, neither these details nor the roof covering is shown on the drawing.

The bridge illustrated in Fig. 37 is over the Tygarts River, at Philippi, W. Va. Built in 1852, it is said to have been the scene of the first land

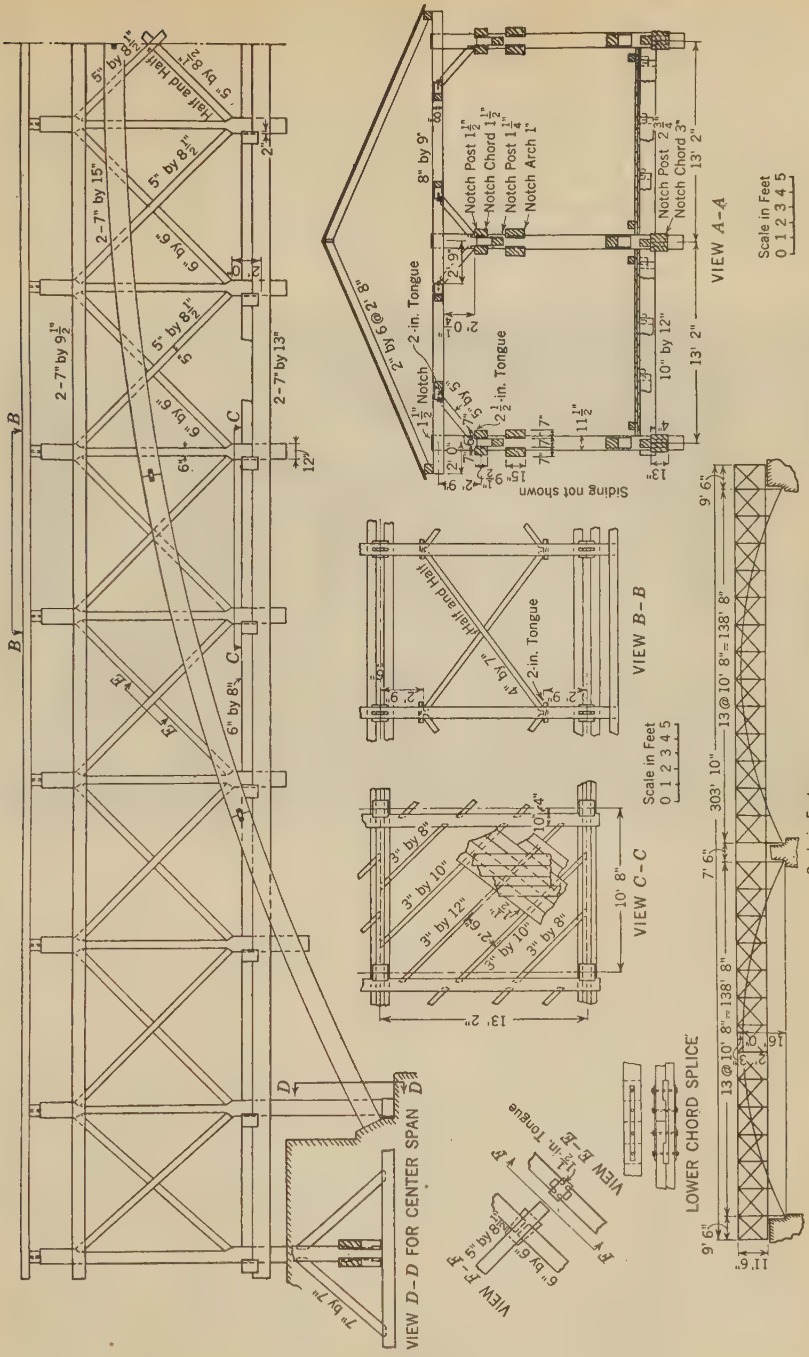
Half and Half
5" by 8½"
5" by 8½"
2"
2-7" by 15"
6" by 6"
5" by 8½"
5
2-7" by 9½"
2-7" by 13"
6" by 6"
B
B
C
C
6½"
12"
E
E
6" by 8"
D
D
VIEW D–D FOR CENTER SPAN
7" by 7"
2" by 6 @ 2' 8"
2-in. Tongue
8" by 9"
1½" Notch
5" by 5"
2½-in. Tongue
Notch Post 1½"
Notch Chord 1½"
Notch Post 1¼"
Notch Arch 1"
Notch Post 2¾"
Notch Chord 3"
13' 2"
13' 2"
10" by 12"
11½"
7"
15"
2' 9"
2' 0"
Siding not shown
VIEW A–A
Scale in Feet
0 1 2 3 4 5
Half and Half
4" by 7"
2-in. Tongue
2' 9"
2' 9"
VIEW B–B
Scale in Feet
0 1 2 3 4 5
3" by 8"
3" by 10"
3" by 12"
3" by 10"
3" by 8"
2' 6"
10' 8"
13' 2"
VIEW C–C
VIEW E–E
1½-in. Tongue
F
F
VIEW F–F
5" by 8½"
6" by 6"
LOWER CHORD SPLICE
9' 6"
13 @ 10' 8" = 138' 8"
303' 10"
7' 6"
13 @ 10' 8" = 138' 8"
9' 6"
11' 6"
Scale in Feet
0 8 16 24 32 40

FIG. 37.—BRIDGE OVER TYGARTS RIVER, AT PHILIPPI, W. VA.

battle of the Civil War. It has two spans, the arches being 138⅔ ft long. The bridge has a peculiar stringer system composed of diagonal 3 by 8-in., 3 by 10-in., and 3 by 12-in. beams, notched half and half into the floor-beams and into 6 by 8-in. timbers placed on, and running parallel with, the lower chords. This type of connection has proved very weak under concentrated loading and has given considerable trouble by horizontal shearing at the ends of the stringers. No lower lateral system was used on this structure, probably for the reason that the stringers, being framed into the floor-beams and pinned as well, were expected to serve as bracing. The excellent type of fish-plate joint used for lower chord splicing should be noted.

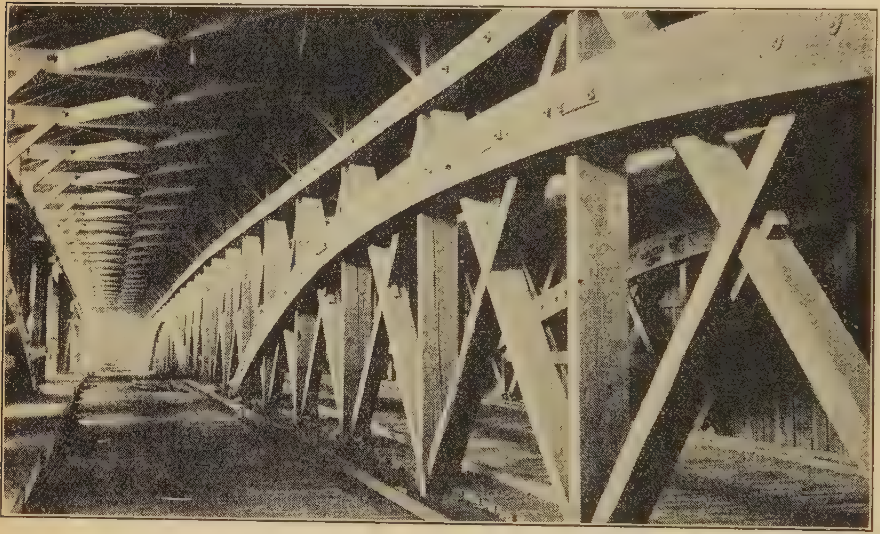

FIG. 38.—INTERIOR VIEW OF PHILIPPI BRIDGE

The bridge was built by Lemuel Chenoweth, of Beverly, Va. (now W. Va.). Mr. Chenoweth was a bridge architect by profession, and it is said that his designs were original with him, and every principle was worked out with mathematical accuracy. He knew beforehand the shape and size of every piece of timber used in the framework of his bridges. The contract for this bridge was let at Richmond, Va. Bidders were present in large numbers with all kinds of models and plans. As far as appearances went, it is said that some of the New England Yankees had models of perfect form and beauty, painted and enameled in the highest art. Mr. Chenoweth's plain wooden model attracted little attention until he placed it on two chairs, one end resting on each, and then stood on his little bridge, and called on the other architects to put theirs to the test by doing the same. This feat got him the job of building the bridge.

Considerable decay of the arches has occurred near the springing lines, but all timbers that have been properly protected are in good condition. After eighty years of continuous use this structure still lines up almost perfectly.

The details of the trusses and arches are quite similar to those of the Waterford Bridge (Fig. 15) described by the authors. Fig. 38 gives a view of the interior of the Philippi Bridge.

The only metal in any of the bridges described was in the form of bolts, which were freely used to hold the various members, and their segments together; but in no case, except the floor-beam connections and the lower chord splices of the Cheat River Bridge, do these bolts take any primary stress. The transfer of load from one element to another was accomplished by direct bearing or by notching. Wedges were used quite freely to take care of shrinkage conditions and wooden pins were utilized in the bracing systems.

PETER T. LANDSEM,[66] ASSOC. M. AM. SOC. C. E. (by letter).—The authors deserve much credit for their thorough investigation of the development of wood bridges in the United States. However, the paper gives the impression of being an epitaph on wood bridges, which is not deserved. Certain developments in the more efficient use of lumber both in Europe and in the United States indicate that the world is probably on the threshold of a renaissance in wooden bridge construction.

In general, information on the period from the collapse of the Roman Empire up to 1500 A. D., seems to be lacking in the authors' treatise. Improvements were small, which is astonishing in view of the highly developed church architecture, both Romanesque and Gothic. Most bridges were simple trestles. An outstanding example was the one built over the Rhine, at Basel, Switzerland, in 1225, and first replaced in 1903.

It might be of interest to state that Palladio did not consider himself the inventor of the truss. He refers to a truss with parallel chords, verticals, and single compression diagonals found in use in Germany by Picheroni de Mirandola.

The authors state that "covered or roofed-in bridges were distinctly not a feature in European practice," yet there is evidence that covered bridges were known in Germany and Switzerland long before the famous Ulrich Grubenmann and Josef Ritter became active in bridge construction early in the Eighteenth Century. One of the oldest of such bridges from the Middle Ages, still in existence, is the picturesque "Kapellbrücke" over the Reuss, in Luzerne, Switzerland, built in 1333. Similar bridges were built in the Sixteenth Century with spans up to 22 m. In 1658, a covered bridge with a span of 32 m was built over the Rhine, at Stein-Säckingen, Germany. Dr.-Ing. K. Schaechterle,[67] has called attention to a number of covered bridges, and advice concerning the construction of covered bridges has been published by L. Chr. Sturm;[68] J. Leupold;[69] Johan Wilhelm;[70]; and Johann Vogel.[71]

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[66] Asst. Engr., National Committee on Wood Utilization, U. S. Dept. of Commerce, Washington, D. C.

[67] "Holzbrücken," 1927.

[68] "Architectura civili-militaris," 1719.

[69] "Theatrum pontificiale," 1726.

[70] "Architectura civilis," 1682.

[71] "Die Moderne Baukunst," 1708.

Developments in Europe in the Nineteenth Century.—The authors of this treatise do not trace the development of wooden bridges in Europe after the middle of the Eighteenth Century, and do not cite any examples of such bridges built in the latter part of the Nineteenth Century in the United States.

A brief summary of the development in Europe may be of interest. About 1800, Funk, in Germany, applied to bridges a wooden arch that had been used for roofs by the French architect, de l'Orme, in the latter part of the Sixteenth Century. The arches were made of boards on edge, cut to shape, and nailed and doweled together. This system was not satisfactory, however, because in shaping the boards the grain of the wood was cut too much and because water entered so easily into the vertical joints.

In 1809, Wiebeking, another German, used for bridges the system generally attributed to the French Colonel Emy, although not in use by him until 1819. The boards were laid flat and nailed and bolted together. This type of arch, however, was too flexible even when braced. Load tests on both de l'Orme's and Emy's systems were carried out by the French military engineer, P. Ardant.[72]

Although the railroads increased the demand for bridges early in the Nineteenth Century, no significant improvements originated in Europe. It was left to the United States to develop the combined arch truss into the framed trusses of Long, Howe, and Pratt. From the middle of the century the art of wooden bridge building went into a decline. This must be attributed mainly to lack of knowledge concerning the properties of wood and concerning reliable framing methods. Engineers of that day lost interest in wood construction. In fact, the only new idea that has come to the writer's attention are the gallow bridges in Trondhjem, Norway.

Modern Developments in Wood Construction.—New interest in wood construction was created about 1900 by carpenter-contractors who, having acquired some technical knowledge, put up structures that caught the interest of engineers. The engineers have carried the developments further. Therefore, in Germany to-day, they speak about "Ingenieur-Holzbau," or engineering wood-construction.

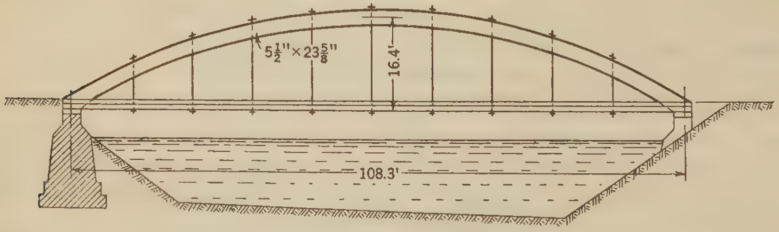

FIG. 39.—SECTION OF FOOTBRIDGE OVER THE WIESE RIVER AT BASEL, SWITZERLAND

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[72] "Theoretischpraktische Abhandlung über Anordnung und Konstruktion der Sprengwerke von grosser Spannweite," von A. v. Kaven, 1847.

Stephan made Emy's arch stiffer by building it like a truss of curved laminated chords, doweled together, and braced by cross-diagonals. This system, however, was used only for roof trusses. The railway station in Copenhagen, Denmark, is a good example.

Otto Hetzer introduced his laminated glued-wood arch in 1907. It consists of boards laid flat, curved, and glued together, the glue being water-resistant and having at least the same strength as the wood. An early example is the foot-bridge over the Wiese, at Basel, Switzerland (see Fig. 39), which was built in 1910. The roadway is suspended from the two-hinged arches, 33 m (108.3 ft) in span. It was computed for a load of 350 kg per sq m (72 lb per sq ft) and cost 6 200 francs, which was only one-half the nearest bid in steel. Th. Gesteschi[73] and H. Lewe[74] mention other arch bridges.

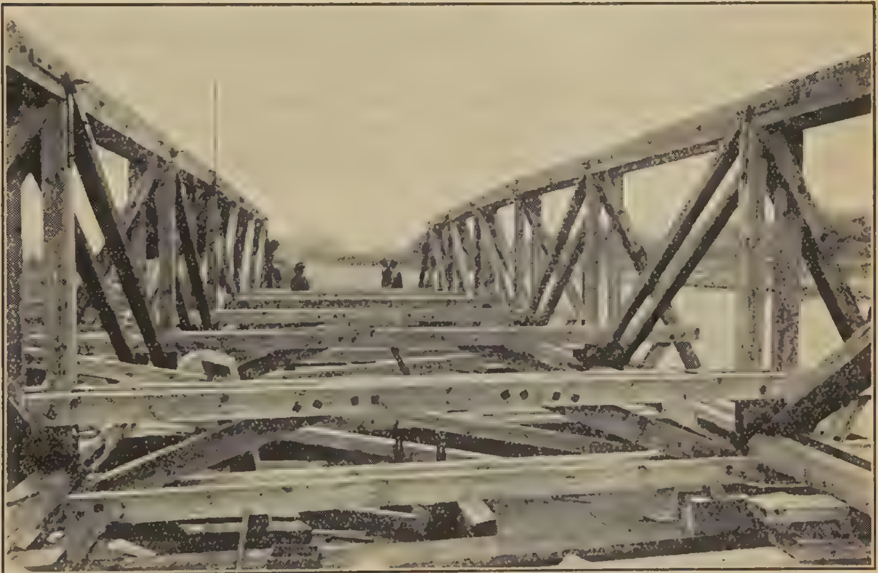

FIG. 40.—VIEW OF CONSTRUCTION BRIDGE OVER THE SPREE, BERLIN, GERMANY; CLOSE-UP OF ONE SPAN

During the World War the simple truss was improved and the trussed frame was introduced with the use of new and more efficient joining devices. Modern European literature refers to many wood bridges. A covered bridge over the Neckar, at Thalhausen, Germany, was built in 1924. It has three spans, 62.3 ft long and two spans, 32.8 ft long. Others are: A suspension bridge at Sulitjelma, Norway, 266 ft long (maximum span, 177 ft); a pipe-line suspension bridge in Finland, with four spans of 167 ft each and a total length of 984 ft; a footbridge at Neumünster Railway Station, in Germany (maximum single spans, 131 ft, and total length, 541 ft); and a military road bridge in Norway (span, 85 ft). Well known foreign sources of information are: Th. Gesteschi;[75] C. Kersten;[75] and A. Laskus.[76]

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[73] "Der Holzbau."

[74] "Neuere Brückenbauten in Holz," *Bauingenieur,* 1921, Vol. 17.

Introduction of Modern Wood Construction Methods in the United States.—It has been recognized that the chief drawback in the use of wood to-day in bridge construction has been the inefficient joint. In bolted connections the safe loads determined by the various handbook methods differed widely. This situation, fortunately, has been relieved by an extensive series of tests[77] carried on for years at the Forest Products Laboratory, U. S. Department of Agriculture. The Engineering Staff of the National Committee on Wood Utilization, U. S. Department of Commerce, in co-operation with the Forest Products Laboratory, U. S. Department of Agriculture, has been investigating still better joining devices which, as tests show, may increase the load capacity from two to six times (and even more) than that of an ordinary bolted joint.

Fig. 40 shows a bridge in which modern connectors are used in the joints. These joining devices or connectors, comprising various types of rings, plates, and disks embedded in the sides of timbers (see Fig. 40), are used for transmitting the load from one member to the other. Fig. 41[78] is the view of a joint composed of modern connectors developed in Germany.

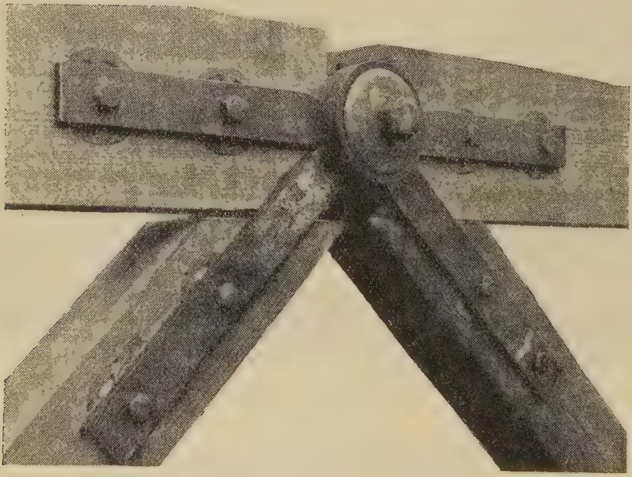

FIG. 41.—VIEW OF JOINT COMPOSED OF MODERN CONNECTORS

In concluding, the writer would like to call attention to the fact that the State of Oregon is building timber highway bridges, using the money thus saved for better road surfaces.

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[75] "Freitragende Holzbauten."

[76] "Hölzerne Brücken."

[77] *Bulletin No. 332*, "The Bearing Strength of Wood Under Bolts," by George W. Trayer, 1932.

[78] "Modern Connectors for Timber Construction," by Nelson S. Perkins, Peter T. Landsem, and George W. Trayer, *Bulletin No. 24*, National Committee on Wood Utilization, U. S. Dept. of Commerce.

J. K. Finch,[79] M Am. Soc. C. E. (by letter).—One or two books on American timber bridges have appeared in recent years, but this paper is the first to record and discuss some of the interesting technical details involved in early timber bridge construction. While the men who built these bridges were simply self-taught Yankee carpenters, their abilities were of the first order and from their labors the truss bridge, a typically American contribution to engineering progress, was evolved.

The writer has no details to add to the record of these master builders as given by Messrs. Fletcher and Snow. There are, however, a few notes that might be of interest on the place this timber-bridge era occupies in the great epic of engineering achievement.

The authors mentioned the lack of bridges in ancient Greece. The same observation would hold for Egypt and Mesopotamia. This was not due to any lack of ability on the part of the Greek architecton, or master builder. (The name, architect, is derived from this title but it is preferable to translate architecton, and the later Roman architectus, as master builder, for, until the introduction of a new technique with the advent of gun-powder in the early Renaissance, the present professions of civil and military engineering and architecture were all included in this single name). Topography was a most important element in shaping the course of engineering development. Thus, the Nile and the Tigris-Euphrates furnished the transportation routes for these two most ancient civilizations. Roads, and hence bridges, were not required. Again, Greece has been described as a huge, bony hand stretching out into the Eastern Mediterranean and Aegean Seas. The construction even of trails across her rock-ribbed peninsulas was difficult. A unified Greek nation was topographically impossible. Roads, as an important element of communication, were not to be thought of. The Greek City States turned to the sea. The first naval battle of the world was fought by the Greek and Persian fleets at Salamis, and the Greeks naturally became the first of ancient harbor builders.

The topography of the Roman peninsula, on the other hand, made it physically possible to consolidate this great area and its many tribes into a single nation from the Alps to the heel of Calabria. This opportunity was taken advantage of by the strongest tribe, the Latin race, and the construction of roads, and necessarily of bridges, became an inevitable, a basic element, to this consolidation. The Alps shut off France from Italy, but France also was consolidated by another great Roman road and bridge-building development, while communication between the seaports of Southern France and Northern Italy furnished the connecting link.

Centuries later, in the United States, economic pressure from the seaboard States, the struggle of each State to secure trade advantages from the expansion westward which followed the Revolution, combined with the topographic problems involved, made the American canal and railroad era, and its accompanying bridge development, inevitable. It was not only Yankee ingenuity, or the force and initiative of a pioneer people, that made the United States the greatest nation of bridge builders the world has ever known. Topography and economic pressure have forced the people of this country to be ingenious and progressive in this particular field.

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[79] Renwick Prof. of Civ. Eng., Columbia Univ., New York, N. Y.

It is also interesting to reflect on some other factors and forces that led American engineers to develop the truss bridge.

Palladio's famous timber truss of about 1540, mentioned by the authors, is well known. The first metal truss, however, had been built centuries earlier, under the Roman Engineer-Emperor Hadrian, to support the portico of the Pantheon at Rome. Undoubtedly, the truss form was used by the Greeks and, later, by the Romans in many roof constructions. Timber, however, was recognized as temporary, and the fire hazard was also considered. Hadrian's truss, for example, was of bronze and was clearly a pioneer attempt at fireproof building construction. (It was removed and melted for making cannon under Pope Urban VIII in 1625, but early drawings show its form and details.) Timber bridges, although undoubtedly used, were replaced by fireproof, durable stone arches whenever economic conditions warranted permanent construction. Hadrian's effort to build framing in metal was interesting, but of no economic importance because metal was too costly. In fact, the truss, an invention in framing, did not emerge, was not discovered or invented, until a civilization arose which was based almost entirely on timber construction. Colonial America was such a civilization.

David Stevenson, the British engineer, writing[80] in 1838 defended the use of timber and temporary constructions by American engineers, a practice which had been criticized by European workers. He pointed out that labor was scarce in the States, funds were not available for costly stone constructions, and the unknown future requirements as to loads and sizes of structures made temporary construction advisable. Construction in timber, therefore, represented not only economical design, but also good engineering and business foresight.

How much such pioneers in this field as Palmer, Burr, and Wernwag were indebted to earlier European bridge builders for their basic ideas, will probably never be definitely known. "Copying" may not have occurred at all. Indeed, books were scarce, and American Colonial carpenters would not be expected to have or see them. This point, however, is unimportant. These men and their associates, as well as earlier workers, apparently had no very clear conceptions of truss action, or at least did not have full confidence in such framing. Their bridges were "statically indeterminate" combinations of arches with "stiffening" trusses. Nevertheless, the germ of the truss principle existed in their work, and ultimately the truss emerged from their composite structures as a form capable in itself of carrying the full load. Town, Long, and, finally, Howe (1840) saw this element in the work of Palmer, Burr, and Wernwag, and the truss was thus brought forward as a distinct and separate structural form.

During this period of truss evolution, British engineers stuck stubbornly to their metal arches, girders, and "tubular" bridges. American engineers, following Haupt and Whipple, began to analyze the stresses in trusses. There was an era of patented trusses in which Pratt, Warren, Bollman, and others contributed to the evolution of economic truss forms. The timber and wrought iron was translated into the cast and wrought-iron construction of the Sixties, and then into the all-steel truss of the Seventies and Eighties. Finally, the engineers of the present day have contributed the great alloy steel spans of the modern highway era.

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[80] "Sketch of the Civil Engineering of North America," pub. by John Weale, Lond., 1838.

The work of the early American timber bridge builders, therefore, represents only one step in the history of a great development. Their contribution to engineering progress, however, entitles these pioneer American bridge engineers to a prominent place in the annals of a great profession.

A. A. EREMIN,[81] ASSOC. M. AM. SOC. C. E. (by letter).—A number of ancient wooden bridges have been described (in Russian) by Mr. L. Nikolai,[82] a reference to which should be made in connection with this paper. For example, he shows the Trajan Bridge over the Danube River at the Iron Gates built by Apollodorus about 109 A. D. This is the bridge to which reference was made by the authors. The clear span was 118 ft, and the stone piers were carried 20 ft below water level. The Trajan Bridge is known as the first wooden arch bridge.

A freedom of the creative imagination at the beginning of the Nineteenth Century is well illustrated by Mr. Burr's bridge,[31] at Schenectady, N. Y., built in 1808. The curved chords were formed of eight 4 by 14-in. planks, spiked, and bolted together. After twenty years of service the bridge was reinforced with additional intermediate piers built in the middle of each span, and, in 1873, the structure was replaced by a metal bridge.

Among the notable wooden bridges built in Russia, Mr. Nikolai has described the "rainbow" arch bridge with suspended deck and a clear opening of 257 ft, spanning the Wepr River at Ivan Gorod Fortress.

In crossing navigable rivers the Russian builders used a removable bridge with draw-spans that were easily opened. The bridge was removed in winter when the river was covered with ice, and in spring, to permit a heavy mass of floating ice to pass at high water.

A primitive type of removable bridge is the floating type of structure. The deck of a floating bridge built in Riga (Fig. 42), was formed of planks 3 in. thick on 6 by 6-in. joists spaced at 3-ft centers and placed on 6 by 12-in. beams spaced at $3\frac{1}{2}$-ft centers. The beams were bolted to the 13 by 13-in. floating longitudinal beams. The average life of this type of floating bridge was about ten years. Later, the floating bridges were replaced with pontoon bridges, the spans being carried on wooden or metal barges.

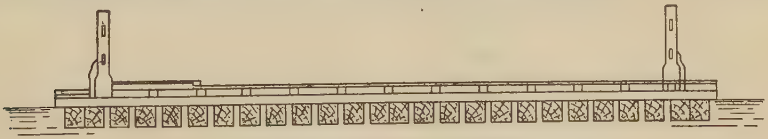

FIG. 42.—FLOATING BRIDGE

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[81] Asst. Designing Engr., Bridge Dept., State Highway Comm., Sacramento, Calif.
[82] "Bridges," by L. Nikolai, Russian Edition, 1901.

AKSEL ANDERSEN,[83] M. AM. SOC. C. E. (by letter).—The following data on wooden railroad bridges built in Norway between 1860 and 1875 is of interest in connection with this instructive paper on the development of wooden bridges. During that period the Norwegian Government constructed several important railway lines, including the northern half of the railroad between Oslo and Trondhjem, and several lines in the southeastern part of the country.

A great variety of conditions were encountered, involving the bridging of numerous rivers and deep valleys in mountainous country, with poor transportation facilities. Skilled carpenters and excellent pine timber were available, however, at low cost, and the selection of wood as the principal building material was natural. Stone or iron bridges would have been much more expensive and would have required a longer construction period.

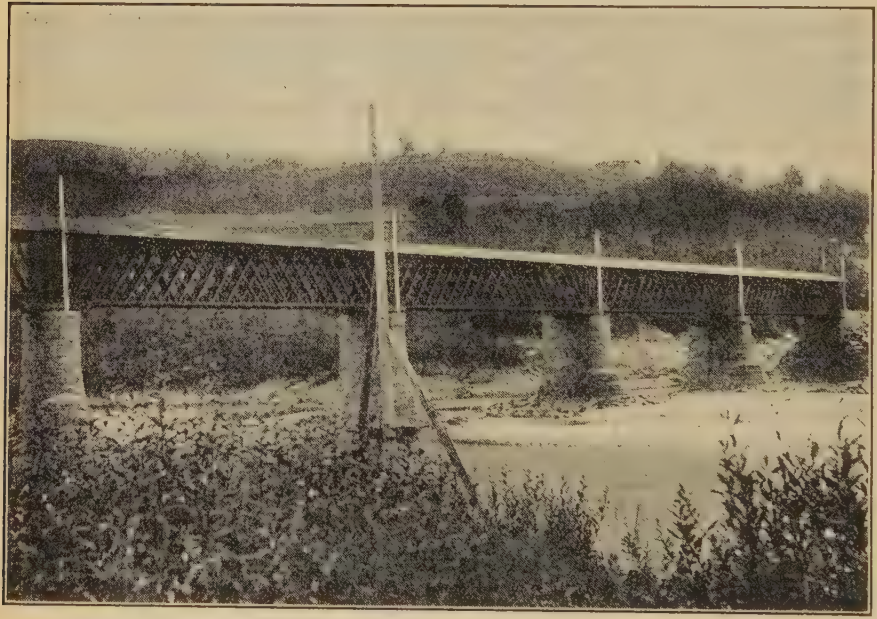

FIG. 43.—VIEW OF GULFOSS BRIDGE

In the lowlands the pile trestle was a common type, with spans as great as 40 ft. Compound keyed stringers with queen or king-braces were used for the longer spans. Practically all structures of this type were built of round, or half-round timber, and the joints were held together with iron screw-bolts.

For river crossings and high viaducts, with spans up to 100 ft, Pratt trusses and Howe trusses, with one and, in a few cases, with two systems of web members were used. Fig. 43 shows a bridge (Gulfoss Bridge), with four continuous-truss spans of 100 ft and several shorter spans. The continuous trusses are of the original Howe type with two web systems, very similar to the trusses of the bridge over the Connecticut River, at Springfield, Mass., shown in Fig. 24. The only apparent difference is the absence of braces against the piers below the lower chord. The same system of continuous 100-ft spans, with two systems of web members, was used in two other railroad bridges, and one of them (the bridge over the Gula River, at Stören), is still in use (1933) after nearly seventy years of service. (In this connection note the statement by Messrs. Fletcher and Snow under "The Howe Truss," that "it is not known to the writers that there has been another bridge built of the Howe type with webs of two systems"). The Gulfoss Bridge and several other through railroad bridges were protected by roofs, extending about 6 ft outside the trusses, with no side coverings.

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[83] With Ash-Howard-Needles & Tammen, New York, N. Y.

Deck spans with simple Howe trusses, or Pratt trusses, supported by timber piers on masonry bases, were used in a number of high viaducts. The Dröja Viaduct, shown in Fig. 44, had one 100-ft, and seven 45-ft, truss spans of the Pratt type. The upper chords and the post were made of wood, but the diagonals and the lower chords were of iron. The diagonals of the 100-ft span consisted of two rods varying from 2⅛ in. in diameter at the ends to 1 in. at the center. The diameter of the threaded ends of the rods was ¼ in. larger than that of the rod; the threaded sections (6 in. long) were attached to the rods by welding. The welds were, obviously, points of weakness.

The wind system consisted of timber crosses in the plane of the upper chords with iron tie-rods at the panel points, and timber cross-frames at each panel point with iron tie-rods at the lower chords. The writer recalls an incident, in about 1916 or 1917, that proved the efficiency of this type of wind stiffening. One of the iron diagonals in a Pratt truss of a high timber viaduct failed, and several trains had probably passed over the viaduct before the break was detected. The only reason why the structure did not collapse was that the load was carried by the other truss and the wind system. A few typical details are shown in Fig. 44.

The Dröja Viaduct and other deck bridges and viaducts of the same type, were protected by a wooden roofing, extending about 3 ft beyond the outside faces of the upper chords. The trough-shaped part between the rails was made of longitudinal boards, covered with iron plates in order to reduce the fire hazard. The roofing outside the rails consisted of two layers of 1-in. by 6-in., transverse boards sloping slightly outward. The boards in each layer were placed with a spacing of 2 in., thus overlapping the boards of the other layer by 2 in. The contact surfaces between the two layers were provided with grooves for drainage (see Fig. 44). A similar system was used for wooden floors of highway bridges. The wearing surface consisted of longitudinal planks, and the loads were carried to the stringers by two layers of overlapping and grooved planks.

Simple Howe trusses, with iron rods as verticals, were used in a number of railroad bridges and viaducts. In some of them, the angle blocks were made of wood (usually oak), but more frequently they were of cast iron.

Practically all wooden bridges designed and built by the Norwegian Government Railways in the aforementioned period have been replaced, in recent years, by stone or steel bridges, or have been eliminated by re-location

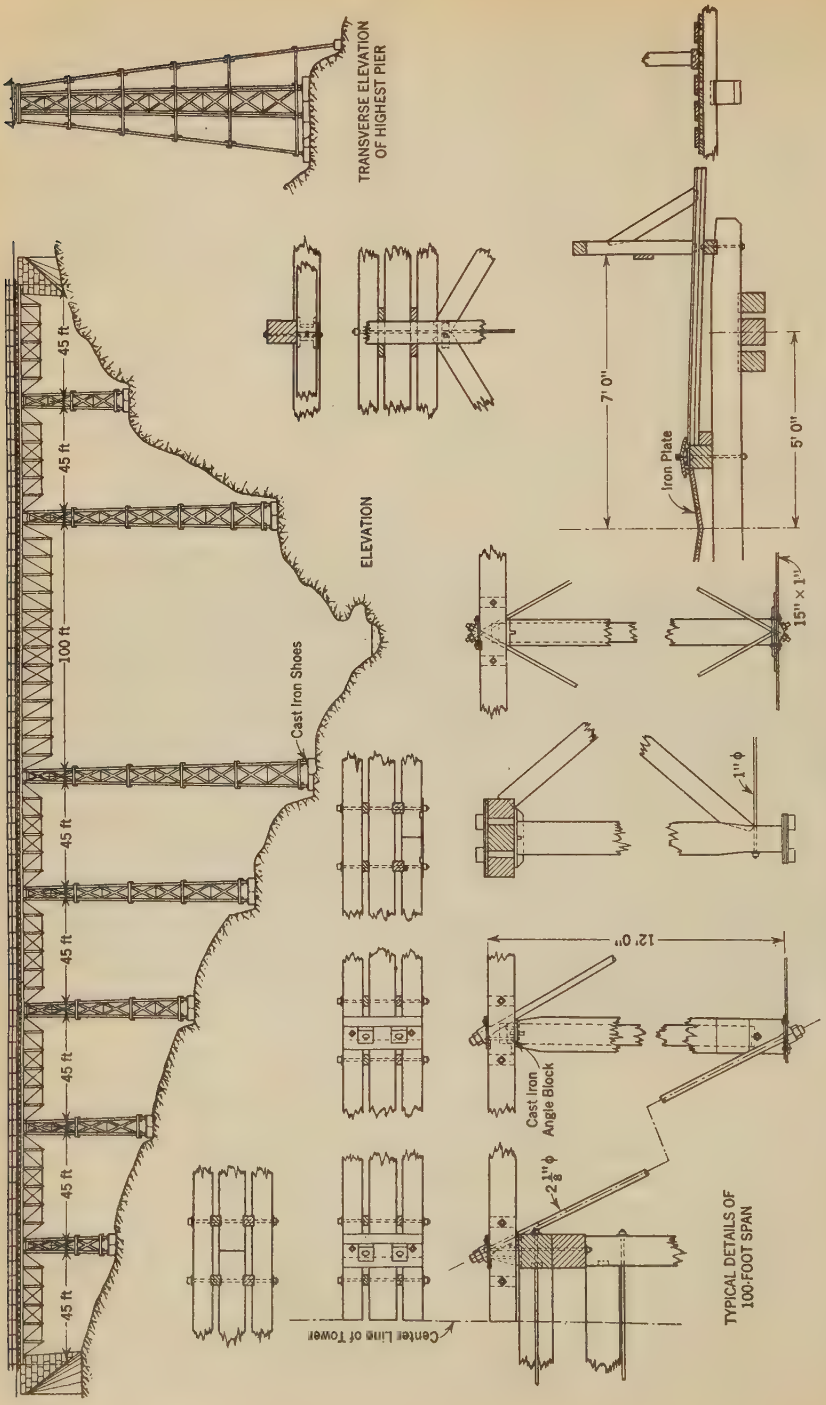
TRANSVERSE ELEVATION OF HIGHEST PIER
ELEVATION
45 ft
45 ft
100 ft
45 ft
45 ft
45 ft
45 ft
45 ft
Cast Iron Shoes
Center Line of Tower
Cast Iron Angle Block
2 1/8" φ
12' 0"
1" φ
15" × 1"
Iron Plate
7' 0"
5' 0"
TYPICAL DETAILS OF 100-FOOT SPAN

FIG. 44.—TYPICAL DETAILS OF TRUSS SPANS, DRÖJA VIADUCT

of the lines. Record drawings, based on field measurements, and a number of photographs of the more important bridges have been collected by the Railway Museum at Hamar.

The writer wishes to acknowledge his indebtedness to Hans Tönnessen, Chief Engineer of Bridges of the Norwegian Government Railroads, who furnished drawings, photographs, and other data for this discussion.

William G. Atwood,[84] M. Am. Soc. C. E. (by letter.)—This paper has great historical interest and contains many lessons for engineers in active practice to-day. The interest created by the development of new or improved materials and the very intelligent advertising and salesmanship of their producers have caused the virtues of some of the older materials to be disregarded.

It is the duty of the engineer to build the cheapest structure that he can, which will be fully adequate for its purpose. If he studies this paper carefully he is likely to question whether many times a full timber or a composite structure, when taking into account interest on investment, maintenance, probable obsolescence, and salvage value, is not really a better and more economical structure than one built entirely with the newer materials.

Most of the structures described by the authors were built of timber because it was the only material physically available. This is still true to-day in a few isolated cases. The writer had a part in the construction of the Alaska Central Railway (later, the U. S. Government Railway), on which, in a distance of 4 miles, there was a total of more than 1 mile of timber bridges, part trestle and part Howe truss. This work was done in 1906 and 1907 when no transportation except pack and wagon train was available and no other material could have been used. Examples of this kind are rare in the present day.

Under many conditions timber trestles or spans will be found more economical than concrete or steel if all factors of cost are taken into consideration, especially that of obsolescence or the possibility of future reconstruction or widening. As an example of this there may be mentioned a number of short-span, through, concrete girder highway bridges on one of the most important north and south through highways. These bridges were built a number of years ago and roadway widening became necessary. The old bridges had to be removed and were replaced by others of the same type. If, originally, they had been treated-timber pile bridges, their first cost would have been less, they could have been widened at a small fraction of the cost of the work actually done, and the treated-timber bridges would have lasted as long as bridges were required in these locations.

This paper not only shows the great ability of the preceding generations of engineers in using the structural materials available to produce bridges that were not only adequate for the loadings of their time, but capable of safely carrying much greater loads, but it also shows the inherent value of the material itself. Timber bridges carrying loads several times those for which they were designed for periods of 50 to 100 years cannot by any stretch of the imagination be called temporary structures.

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[84] Cons. Engr., New York, N. Y.

When giving credit to their professional predecessors engineers should remember that they did not have the information that is available at the present day even regarding such an old structural material as timber; nor was treated timber available in any quantity until the last fifty years. As a result of the work of the U. S. Forest Products Laboratory, the Department of Commerce, and the various associations of timber producers and timber treaters, together with the engineering societies, there is available for the present-day designer information that permits him to select his timber and treatment with a certainty as to strength and durability that is not exceeded in the case of other structural materials. Engineers of this generation, therefore, may build stronger and more durable structures with less timber than their predecessors. The paper under discussion shows what the past generation could do.

It is a fact which will probably be questioned by few who are conversant with present practices, that frequently advantage is not taken of the information now available. The Highway Department of one important State is now using timber grading rules and specifications written thirty years ago, and an important city is using creosoting specifications nearly as old. This is a waste of money.

Men like Timothy Palmer whose bridges are described in the paper, were in advance of their time. This generation of engineers may well ask themselves whether in their frequent disregard of the oldest and one of the best of the available structural materials, they are worthily "carrying the torch of progress."

H. FUGL MEYER,[85] ASSOC. M. AM. SOC. C. E. (by letter).—A brief statement as to the contribution of the Chinese to the art of bridge building is of importance in establishing the origins discussed in this interesting paper. The Chinese character, "chiao," denoting a bridge, contains in its components the idea of a lofty wooden construction. The character was formed about 1003 B. C., thus lending authority to legendary history, which records the information that the Chinese had undertaken the construction of fairly large wooden bridges at a very early date. From the Confucian annals and from the history of the Near East it is known that in a number of Asiatic countries wooden pontoon bridges of considerable length were in regular use before 600 B. C.

At that early date there were a number of communities with a fairly high civilization in the part of Central Asia that now (1933) forms the western border of China. Because the turbulent and wide mountain streams in this territory made all inter-communication difficult, Nature made the inhabitants bridge builders long before written records of their kingdoms begin.

A number of interesting wooden cantilever and suspension bridges are still to be found in these regions. Whenever the width of the stream is less than, say, 130 ft, a cantilever appears to be preferred. Suspension bridges of plaited bamboo ropes are used for spanning streams as wide as 250 ft.

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[85] Mgr., Free Port of Copenhagen, Copenhagen, Denmark.

Multiple-span bridges of both types of construction are still extant, the longest being a suspension bridge of 700 ft.

Chinese bridges are generally the principal gathering place for the inhabitants of the neighboring town, and, therefore, they are often covered with a roof or are fitted with supports for a light roof of bamboo matting as a protection against sun and rain. Fig. 45 represents a bridge in the small Tibetan-Chinese Kingdom of Muli. It will be noted that two heavy stone-filled wooden caissons lend stability to the cantilevers. The central part of the span can be removed when the King of Muli finds it necessary to block this entrance to his realm.

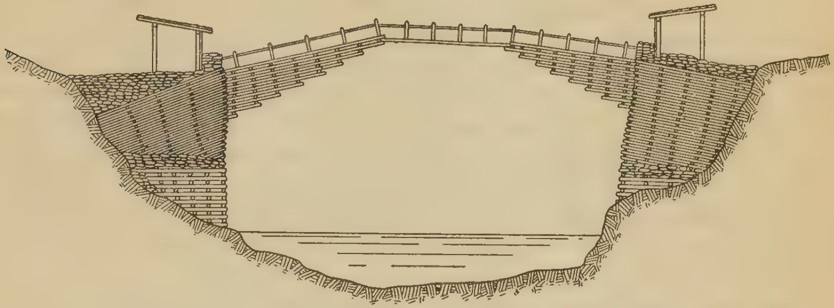

FIG. 45.—CANTILEVER BRIDGE IN THE KINGDOM OF MULI.

W. E. HOWLAND,[86] ASSOC. M. AM. SOC. C. E. (by letter).—In an "Encyclopædia or Dictionary of Arts, Sciences and Miscellaneous Literature," printed in Philadelphia, Pa., for Thomas Dobson, in 1798, and in the "Supplement" published in 1803, several articles appeared relating to wooden bridges, namely, "Arches," "Centering," "Carpentry," "Roofs," and "Strength of Materials," all related one to another with numerous cross-references. This twenty-one volume compilation[87] offers an opportunity for the student of the history of the development of technology to determine what was known about a particular science at a given period, and, in this case, it shows what knowledge might have been carried over from Europe to America. Such a medium as this could have brought to the early American bridge builders the knowledge gained by their European colleagues, not only in building wooden bridges, but also in the science of the equilibrium polygon and in countless practical details of wood construction for carrying heavy loads.

Although, as the authors have pointed out, the European wooden bridges were not especially noteworthy at this time, certain other Continental wooden construction appears to have been developed to a high degree of perfection before the era of American wooden bridge construction, for example, the wooden centering of Perronet and the wooden roof trusses of Sir Christopher

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[86] Asst. Prof. of Civ. Eng., Purdue Univ., Lafayette, Ind.

[87] Reprinted from "Encyclopaedia Britannica," or a Dictionary of Arts, Sciences and Miscellaneous Literature," Third Edition, printed for A. Bell and C. Macfarquhar, Edinburgh, Scotland, 1797.

Wren. The writers of this encyclopædia assembled this knowledge and made it available in a convincing and understandable form for the potential bridge builder. It seems inconceivable that a reader of these articles would not have profited immensely both in obtaining a clearer mental picture of the principles involved and the manner in which the forces distribute themselves in a complicated frame, but also in adding to his knowledge of practical details of construction.

The article on "Strength of Materials" is a mine of information to the historian of science. Its author considers tension, compression, and shear and the resistances of various materials (particularly wood) thereto. He considers Euler's column theory, and he derives a practical rule for estimating the strength of a beam in bending which is in close agreement with modern ideas; that is, he assumes that the strength varies directly with the breadth of the beam and with the depth squared, and that a beam loaded in the center requires a strength that is proportional to the span and to the load. All the theory presented is discussed in the light of practical knowledge and is translated into rules-of-thumb for the use of the artisan.

The article on "Roofs" contains a discussion of improper practice in joining wooden members, and it ends with the maxim of Perronet, namely, "to make all the shoulders of abutting pieces in the form of an arc of a circle having the opposite end of the piece for its center."[88] Then, to connect this subject of roofs with other phases of wood construction, the author adds this sentence,[89] "much of what has been said on this subject may be applied to the construction of wooden bridges and the centers for turning the arches of stone bridges."

The work of Perronet is discussed in the article on "Centering,"[90] with critical insight and yet with admiration. This section treats of the history of the development of the center to produce the stiffness needed for maintaining the shape of the stone arch in various stages of its construction. The author commends the center used by Mr. Robert Mylne (1734-1811) for the Blackfriars Bridge, at London, England. This bears a striking resemblance to a Bollman truss, with the compression members and tension members reversed. The construction consists of a number of pairs of struts, each one beginning at an abutment and terminating at a joint on the outside of the centering, the entire framework being joined together to form a structure of great rigidity.

In the article on "Carpentry,"[91] the author again refers to the work of Perronet. He states, for example, that:

> "The strongest of all methods of piecing a tie-beam would be to set the parts end to end and grasp them between other pieces on either side * * *. Mr. Perronet used it for the tie-beams or stretchers by which he connected the opposite feet of a center which was yielding to its load and had pushed aside one of the piers above 4 in."

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[88] "Encyclopædia or Dictionary of Arts, Sciences, and Miscellaneous Literature," 1798, Vol. 16. p. 476.

[89] *Loc. cit.*, p. 480.

[90] *Loc. cit.*, Supplement, 1803, Vol. 3, p. 234.

[91] *Loc. cit.*, p. 212.

The author then describes the use of scarfing employed by Perronet for eliminating bending in the bolts that connect wooden members. Two full pages of these and other joints are included, and two pages of illustrations of roof trusses, one with a clear span of 80 ft, and a sketch of the walking beam truss used by Watt and Boulton on their steam engine. It has tension members for the top chord.

The author on "Centering" discusses disapprovingly the wooden arch built by Grubenhamm, at Wittingten, Switzerland.[92] He then refers with admiration to a "bridge over the Portsmouth river in North America, * * * more than 250 ft in length * * * of several parallel arches of beams." The inventor said to be a Mr. Blodget, is quoted as saying that he found the strength of this bridge so great that he could with perfect confidence make one of four times the span. Could this be the Piscataqua Bridge of Timothy Palmer built in 1794 and referred to by Messrs. Fletcher and Snow?

Finally, in recommending the best form for a wooden bridge, the contributor of this article returns to the ideas of the roof truss and the arch centers, concluding with these significant words:[93]

> "The reader must perceive that we have now terminated in the construction of the Norman roof. We indeed think it the best general form, when some moderate declivity is not an insuperable objection. When this is the case, we recommend the general plan of the centering of the bridge of Orleans. We would make the bridge (we speak of a great bridge) consist of four trusses; two to serve as the outsides of the bridge, and two inner trusses, separating the carriage-way from the foot-paths. The road should follow the course of the lower polygon, and the main truss should form the rails. It might look strange; but we are here speaking of strength; and evident, but not unwieldly, strength, when once it becomes familiar, is the surest source of beauty in all works of this kind."

HARRY K. ELLIS,[94] ESQ. (by letter).—The authors state that "as late as 1930, there were between 450 and 500 covered bridges in use." This must be an under-estimate, because in Chester County, Pennsylvania, alone there are forty-four covered wooden bridges at the present time. Nine were torn down in 1932, so that in this County there must have been at least fifty-three covered bridges in 1930. Many of the other counties in Eastern Pennsylvania still have large numbers of such bridges.

Among the bridges still existing in Chester County is one of 100 ft span, built in 1807 and still carrying traffic, and one of 60 ft span, built in 1819. The latter bridge is shored up at the present time. The writer believes that the one built in 1807 must hold the record for long life of Burr truss bridges, since Burr's first bridge was built only three years earlier.

Most of the covered bridges in Chester County are of the Burr type with the wooden arches extending to a height about 2 ft below the eaves. White pine was used in almost every case. Some of the bridges are of the queen-post type, without diagonals in the middle panel, and with the floor-beams hung upon a single bolt which passed through the center of the floor-beam upward through the lower chord and about 1 ft to 1.5 ft higher through the truss post. A notch cut in the post permitted the insertion of a nut into which the bolt was screwed. Usually, this notch was on the outside of the truss and covered with the siding boards so that inspection was difficult.

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[92] "Encyclopædia or Dictionary of Arts, Sciences, and Miscellaneous Literature: Supplement," 1803, Vol. 3, p. 248.

[93] *Loc. cit.*, p. 249.

[94] County Engr., Chester County, West Chester, Pa.

It is true that the floors were the weakest part of these bridges, but the writer has twice seen a perfect compression failure of the arch rings. In both cases the wood broomed out on all four sides of the ring at the point of failure. In both cases, the arch was subject to a greatly increased load due to the bridge as a whole being much out of line.

NELSON J. BELL,[95] M. AM. SOC. C. E. (by letter).—This paper is most instructive, especially since technical literature in the past half century seems to have almost wholly neglected the subject. The general opinion among engineers has come to be that the old wooden highway bridges are fit objects of sentimental interest, but of very little practical worth in this age of steel and concrete. The paper by Messrs. Fletcher and Snow should bring home the fact that with only nominal care these old bridges have served their purpose with almost 100% efficiency over periods of 50 to 100 years, and, therefore, may be worthy of close technical study before it is decided whether one should be further preserved for practical use or should be condemned and scrapped. The mortality of these old bridges has been greatly increased in recent years because of the lack of knowledge concerning them and the consequent offhand opinion that they are unsafe under present-day traffic.

The preponderance of evidence brought out in this paper is to the effect that the bridges built by Wernwag, Burr, and most of their imitators, had an excess of strength everywhere except in their floor systems and lateral bracing, but no one has as yet described a method by which the distribution of stresses in the trusses may be analyzed. An exact analysis is probably impossible, and, in any case, would involve so many unknown factors that it would be highly impractical. In some cases it is possible, however, to arrive at a reasonable analysis of the primary stresses in the members by means of assumptions and simple graphic statics.

Several years ago, the writer was employed to investigate a wooden bridge spanning the Kentucky River at Camp Nelson, about twenty miles south of Lexington, Ky. This bridge was built in 1838 by Lewis Wernwag. It has a single span of 240 ft and is composed of three trusses which are identical in all respects, except that the outer ones are curved horizontally so as to form flat arches, equal and opposite to each other. The outer trusses are spaced about 25 ft, center to center, at the middle of the span, and about 30 ft, center to center, at the ends. The center truss is straight and bisects the bridge, forming two one-way roadways. The general design is so similar to that of the bridge over Cheat River, Preston County, West Virginia (Fig. 35), that both must have been designed by the same man. The Camp Nelson Bridge is much heavier, the arches being composed of six pieces, in two rings of three pieces each. There is also a double web system, the main diagonals being composed of two pieces each, passing outside the arch rings, and the single-piece counters passing between the two arch rings. The verticals have three pieces each, the center one passing between the arch rings and the others passing outside. Thus, the arches are clamped against lateral deflection by the verticals and diagonals. The verticals are dapped over the arches and bolted to them, but the diagonals have no connection to the arches, except by friction. An illustrated description of this bridge, with a tabulation of maximum stresses and an outline of the method of analysis was published[96] in 1928 by the writer and J. K. Grannis, M. Am. Soc. C. E.

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[95] Structural Engr., Dayton, Ohio.

It seems to be characteristic of Wernwag's bridges that they have heavy arches with comparatively light stiffening trusses, and that their ends are anchored to the abutments by tension rods. The Camp Nelson Bridge is composed essentially of three arches, each braced laterally and vertically by a stiffening truss, and the whole tied together with lateral trusses at top and bottom. The points of application of loading on each arch are spaced uniformly along the perimeter of the arch by means of the inclined posts, set normal to the curve of the arch. Dead and uniform live loading, therefore, causes compression in the arch, coincident with its center line. The truss is not stressed except by eccentric loads, under which condition it distributes the loads uniformly to the arch, thus limiting all stress in the arch to compression. The stress distribution can be reasonably compared to that in a suspension bridge, the only difference being that in this case there is an arch in compression instead of a cable in tension. With any eccentric load on the truss, it tends to rotate on the arch. This must be counteracted either by the dead weight of the truss, or by anchoring the ends to the abutments, and it is interesting to note that the trusses of the Camp Nelson Bridge were actually anchored by wrought-iron rods extending from the tops of the end posts down into the masonry.

The trusses by Burr and some others do not lend themselves to stress analysis as readily as those by Wernwag, as they seem to have been designed on the principle of a simple truss aided and strengthened by arches, so that there is no definite place where truss action stops and arch action begins. It would certainly be on the safe side to make the analysis on the basis of a stiffened arch where the arches have adequate lateral and vertical support to prevent distortion, but as the dead loads of these bridges greatly exceed the live loads, it is more practical to assign the dead load to the arches and the live load to the trusses and analyze them separately. In any case, the judgment of the investigator enters largely into the problem, and must supplement the mathematical work.

The main point which the writer wishes to make is that the old wooden bridges should not be condemned and scrapped just because they were not designed with the aid of present-day engineering knowledge and standards. Many of them can be made safe and serviceable for modern traffic by very simple repairs and reinforcement of their floors and lateral bracing systems, at a small fraction of the cost of removal and replacement.

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[96] *Engineering News-Record,* Vol. 100, No. 6, February 9, 1928, p. 234.

ROBERT FLETCHER[97] AND J. P. SNOW,[98] MEMBERS, AM. SOC. C. E. (by letter).—A gratifying interest is revealed by the number of discussions submitted in connection with this paper. These discussions have brought to light historical matter of great value, and have increased the knowledge of timber bridges still doing good service. They have widened the view of the ancient history and of the later European period. That timber bridges have yet a "reason to be" is amply proved; and their use for some decades seems to be possible. These fruitful results of a suggestion made by other members of the Society constitute an important chapter of the history of the profession, which might have been lost beyond recovery.

Concerning Cæsar's military bridge—to meet the objection that the notched blocks, as the only connectors at the end of the bents, might fail by crushing at the sharp edges, or by splitting between the notches—a model of one span on two bents was made on a scale of one-twelfth that of the prototype. An estimated load on the span to include the weight of the large, rough-hewn green timber and a column of troops, four abreast, carrying arms and accoutrements, was computed to exceed 14 000 lb. By scale reduction the equivalent load on one bent of the model should be in excess of 100 lb (1 to 144). The model sustained two heavy men aggregating nearly 400 lb in weight, on two bents; that is, nearly 200 lb on one bent. After separation, the parts gave no indication of over-strain.

Figs. 46 and 47 are two notable views of Timothy Palmer's Bridge at Easton, Pa. (see Fig. 11), taken after the bridge had been in continuous service for 89 years. These photographs were taken in 1894 by Eugene Mowlds, M. Am. Soc. C. E., just one year before the removal of the bridge to make way for a structure capable of sustaining traffic by trolley cars. These views of the exterior are valuable as showing the care which the master builder exercised to produce a distinctive architectural effect, in contradistinction to the barren, cheap-as-possible housing since adopted in covering wooden bridges. Fig. 46 shows the appropriate covering with pleasing lines, harmonizing with the surroundings and giving a sturdy aspect to the structure as a whole. Fig. 47, a view of the Easton end, shows enough adornment to relieve homeliness; and, above the portal, is the name and date to serve as the signature attesting the authorship.

Under the heading "The Howe Truss," the writers mention as unusual types, two Howe truss bridges with an arch on both sides of each truss, and state that they had no knowledge of any other Howe truss bridges so constructed. In correspondence with the writers, F. J. Nevins, Valuation Engineer, of the Chicago, Rock Island, and Pacific Railroad Company, and Frank F. Fowle, of Frank F. Fowle and Company, Consulting Engineers, of Chicago, Ill., have stated that the first bridge over the Mississippi River was built for the Chicago, Rock Island and Pacific Railroad Company, between Rock Island, Ill., and Davenport, Iowa, during the years 1853 to 1856. It consisted of five fixed spans of Howe trusses, each about 253 ft, with arches on both sides of each

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[97] Director Emeritus, Thayer School of Civ. Eng., Dartmouth Coll., Hanover, N. H.

[98] Cons. Engr., Boston, Mass. Mr. Snow died on September 4, 1933.

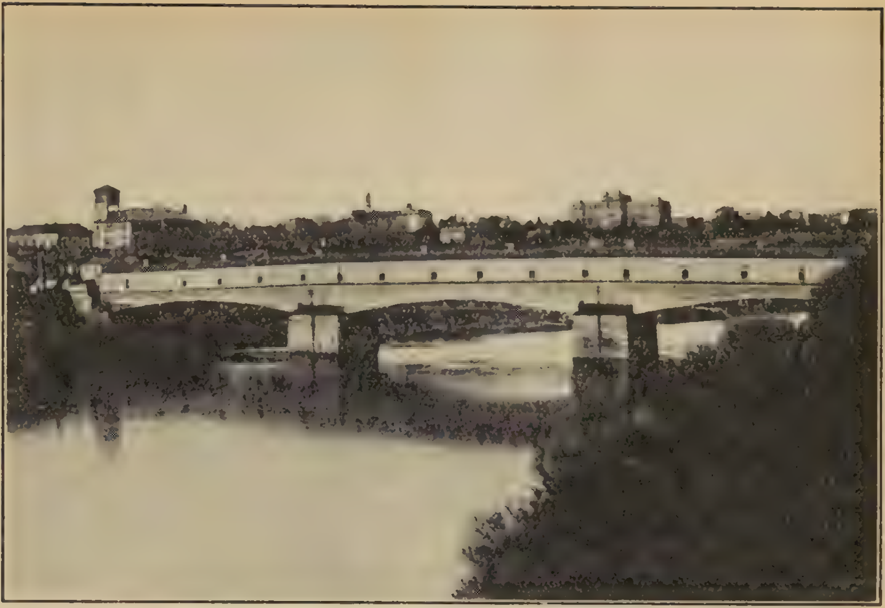

FIG. 46.—BRIDGE OVER DELAWARE RIVER, AT EASTON, PA.

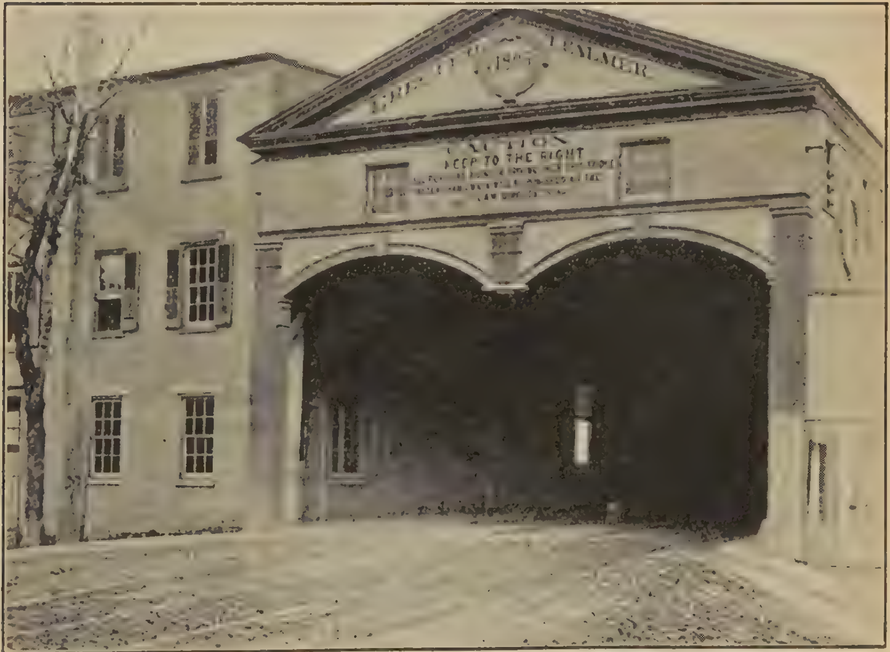
PALMER
CAUTION
KEEP TO THE RIGHT

FIG. 47.—ENTRANCE TO BRIDGE OVER THE DELAWARE RIVER, AT EASTON, PA.

truss, and a draw-span. They also state that a second bridge was built a few years later on the same masonry, of Howe trusses having the unusual feature of curved upper chords, and no auxiliary arches. These are historic bridges because of the strenuous and prolonged opposition of the steamboat owners up and down the river to their construction, and because the unbroken railroad connection thus made across the river gave a great impetus to the settlement of the States beyond.

Questions listed in a letter by J. R. Worcester, M. Am. Soc. C. E., with appropriate answers by the writers, are as follows:

Question 1.—Have not arches been found beneficial to abutments, as well as to trusses, in furnishing lateral pressure where most needed?

Answer.—An arch, of course, tends to assist an abutment in resisting the pressure from the filling at its back, but it should not be depended upon to save a weak abutment. An instance is recorded in which a bulging abutment led to collapse of the arch, breaking it at the crown, and buckling the truss. On the other hand, a case is cited of a **T**-abutment with narrow stem, in which buttress masonry behind the heads was needed when arches were added to the bridge.

Question 2.—What is the reason for the different thicknesses of planks shown in Fig. 17?

Answer.—It is wholly to facilitate construction. The customary way to start an arch is to set up light supports on the falsework to define the curve of the intrados and tack $\frac{7}{8}$-in. boards, which bend easily, to them; after which, 2-in. plank (perhaps not so many as shown), are secured to get a solid support on which the 3-in. or 4-in. planks can be clamped and spiked.

Question 3.—Does the working between parts in combined arches and trusses show itself in loosening of joints, in course of time?

Answer.—In lattice bridges the connection between arch and truss is invariably made by vertical rods through the arch and floor-beams or by needle-beams below the lower chords. Any slackness in these rods is corrected by the screw-threads at their ends. This type of connection is advisable with other kinds of truss, but in the Burr trusses of early days the favorite method was to frame the posts to make seats on the arch. The flexibility of timber is such that no serious working at these joints has been observed by the writers. The creaking observed when a train crosses one of these bridges is normal to wooden bridges. It may be caused by warped sticks coming to a full bearing when a load passes over the bridge. It is probably more pronounced in spruce bridges than in those built of white pine, on account of the greater tendency of spruce to warp.

Question 4.—Can you give any figures as to sag of a wooden truss due to shrinkage, and is this less in a Town lattice than in a Howe truss?

Answer.—Sag in wooden bridges is probably not so much due to shrinkage as to compression of surfaces in contact at joints. Lattice trusses without arches naturally sag more than Howe trusses on account of the great number of joints composed of wooden pins in bored holes. This involves pressure on the rough side-grain of pins as well as a tendency to deflection of the pins. To counteract this tendency it is good practice to build 50% more camber into lattice trusses than into those of the Howe type. Howe trusses, if sagged, can be adjusted by setting up their vertical rods. In the lattice type the sag is cumulative. The unique bridge at North Blenheim, N. Y., shown in Fig. 20, has held its camber from 1855 to the present, although it has no vertical truss rods. This is due, without doubt, to the care used in its framing and the seasoning of the timber before framing as noted by Mr. Morse in commenting on the Ninth Street Bridge, in Pittsburgh, Pa.

Question 5.—What units have you found safe for stresses in bolts and treenails in shear? For direct tension and horizontal shear in spruce and yellow pine? Also, in fiber stress in stringers?

Answer.—As is shown in Mr. Guppy's discussion the size of members depends in some cases on connections rather than on longitudinal stress. Unit stresses that have been found, by experience, to result in good practice are listed in Table 1.

TABLE 1.—ALLOWABLE STRESSES IN TIMBER

Description	Yellow pine	Spruce	Oak pins	Iron
Tension	1 000	800	.....	......
Compression	800	650	.....	......
Bending modified by factor containing depth ÷ span	1 200	1 200	.....	......
Bearing end-grain	1 000	800	.....	......
Bearing side-grain, whole width of timber	350	300	.....	......
Bearing side-grain, small area like washers	400	350	.....	......
Shearing, longitudinal	80	80	.....	......
Bearing	.....	.....	400	......
Shearing	.....	.....	600	......
Bending	.....	.....	1 800	......
Floor-beam hangers; tension, net section	.....	.....	.....	7 500
Lower chord coupling rods; tension, net section	.....	.....	.....	10 000
Lower chord gib-plates; bending	.....	.....	.....	12 500
Arch suspender rods; tension	.....	.....	.....	10 000
Lateral rods, loaded chords; tension	.....	.....	.....	15 000
Lateral rods, unloaded chords; tension	.....	.....	.....	20 000

Bridges built to these units have carried loads, the computed stresses from which are twice these units, without showing any ill effects.

Mr. Seaman shows a 2-track jack-knife draw (Fig. 30). The birth and culmination of this type of draw have been shown[69] in *Engineering News-Record,* by a single-track patent drawing and a photograph of the 8-track draw of the Boston and Maine Railroad Company, over the Charles River channel, at Boston, Mass.

Professor Draffin's notice of the Enoch Hale Bridge is appreciated. It was the first in the United States, that could be called a framed bridge, its predecessors having been of the stringer type, none of greater span than could be negotiated with the magnificent pine spars then available. Mr. Hale secured money from Mr. Geyer, a merchant of Boston, mentioned by Professor Draffin, by a mortgage on his toll bridge. It was a well-paying piece of property which Mr. Geyer had been trying to buy for several years. The mortgage was on short time, and, when its expiration approached, Mr. Hale succeeded in borrowing from his friends to liquidate it. He sent the money to Boston by a messenger who dallied a day at Lowell, and when he tendered the funds to Mr. Geyer, was told that he was a day too late, that the mortgage had expired, that Mr. Geyer had already assumed the title, and that the bridge was his. Such was the law of foreclosure in those days, derived from the Cavalier laws of England, the result being that Enoch Hale lost his noteworthy bridge and became indigent in his old age.

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[69] *Engineering News-Record,* Vol. 108 (1932), p. 93.

Professor Draffin states in his first paragraph that the present concrete successor of the Hale Bridge was built in 1930. The previous lattice bridge was removed in 1930 on account of failure due to a decayed wall block, but the handsome concrete structure was completed and dedicated with fine ceremony in 1932.

The writers concur in Professor Draffin's conclusion that repairs rather than rebuilding were done by Mr. Geyer when the Enoch Hale Bridge was eighteen years old; and that it was done as window dressing in connection with obtaining a mortgage instead of for strengthening the structure. In fact, there is a book extant containing a picture of the village and landscape made from a point at a considerable distance from the bridge, showing its braced stringer outline with the spandrils filled with lattice trim. Probably this trimming with new fences and floor were the extent of Mr. Geyer's repairs. The statement made by President Dwight about the bridge being an obtuse arch has misled several writers. A braced stringer bridge of that magnitude is comparable to a segmental arch of three voussoirs which make obtuse angles with each other. Fig. 8 is an excellent line drawing of the oil painting made soon after the bridge was erected, but the braces were much more sharply inclined than shown, and the right-hand end of the floor should be raised to continue the grade of the other end to meet the side of the gorge higher than indicated. The highway along the mountain side was much higher than the bridge, so that a steep descent was required.

In speaking of the Tucker Bridge (Fig. 18), Professor Draffin calls it of the lattice timber type. This is technically wrong. It was a single-plank lattice bridge of 3 by 10-in. plank, while Fig. 19 is a timber lattice of 6 by 8-in. timbers. Much more of historical interest could be written of this bridge and its contemporary events than is pertinent.

It is gratifying to read in the discussion by Mr. Morse, his appreciation of the splendid workmanship in the bridges built by Theodore Burr. The writers refer to the same characteristic in those built by Timothy Palmer; and indeed it was commented[20] on by the late Theodore Cooper, M. Am. Soc. C. E.

The writers join Mr. Storrs in honoring the early builders who had little to guide them in planning or building their bridges, except their experience and strong common sense. These men and the early builders named in the paper were real engineers. The historians of America have generally ignored them, perhaps because they were regarded as mere carpenters. They were among the "empire builders." They are more deserving of large mention in history and inscriptions on memorial tablets, than many who have achieved less for their country. They are comparable to the "Brothers of the Bridge" who built and maintained the great stone bridges of Europe during the Middle Ages.

The writers can not entirely subscribe to the enthusiasm expressed by Mr. Hogue in the second paragraph of his discussion, as to a recrudescence of timber construction in the country at large; but in regions of primitive growth, as claimed in the paper, they believe a considerable field is open for wooden bridges. The examples of longevity cited for treated lumber (40 years), seems absurd when applied to truss bridges. If properly covered with roof and side boarding 100 years is frequently attained. An example is cited later in this discussion of one, 120 years old, which is still "going strong."

The ring dowels for timber connections mentioned by Mr. Hogue are certainly worthy of consideration. Astonishing examples of their use are illustrated in recent publications of the National Committee of Wood Utilization, U. S. Department of Commerce, Washington, D. C.

The paragraphs of the main paper describing Fig. 16 answer Mr. Oliver's uncertainty in regard to the writers' conclusions as to the adaptability and economy of wooden bridges. Mr. Oliver does not mean that the Selby Bridge, built in 1790, is still in use, as to its original timbers. Stringers bridges last from 5 to 20 years, depending on latitude, character of lumber, and exposure to the weather, but their members are readily renewed, and hence the bridge, as an adjunct of transportation, lasts perpetually. Preservative treatment will double the durability of their members.

Regarding Pons Sublicius, mentioned by Mr. Oliver, the writers do not think any published picture of its construction is authentic.

The writers are greatly indebted to Professor Laurson for the items relating to early European bridges, which he cites. It is interesting to learn that lattice bridges were built on various railroads in England and Ireland previous to 1846. Ithiel Town's patent for this type of bridge was granted in 1820, and it is on record that he visited England about 1840. No doubt he was successful in selling rights to build by his patent. Regarding "assembling" bridges on land before erection in place: If Professor Laurson means assembling them erect, it is seldom, if ever, done. The only practical way to build lattice bridges and others, such as the Burr and Long types, in which the web members pass through the chord, is to frame the trusses lying flat on skids on shore, or on falsework, if wide enough, and traffic is suspended. Holes for pins and bolts are bored in this position and, when complete, each stick is carried to its place on the falsework and assembled erect. The chords of Howe trusses are framed on the falsework and the trusses erected complete after the lower chords are in place. The practice has been to fabricate and erect these bridges as one job; whereas in metal bridges the two phases of the work are widely separated. This distinction may disappear in the Machine Age, supplemented with new methods of making timber connections;

but it is incredible that the building of wooden bridges should become common in regions where organized fabrication shops are within economic reach of the locus of the bridge.

A wrong impression is conveyed by Professor Kirby's reference to the span of the Schaffhausen Bridge (Fig. 7). It consisted of two spans of 171 and 193 ft. The writers have no data showing the span of the McCalls Ferry Bridge, but mention several of much greater span than the Schaffhausen, including some built previous to 1814. (See Wernwag's "Colossus" of 340-ft span, built in 1812 (Fig. 12), and one at Wittingen, Germany, 390-ft span, built during the third quarter of the Eighteenth Century.)[17] It is an interesting fact that a complete model of the Schaffhausen Bridge is still treasured in the town 134 years after the destruction of the bridge by a French army. This is without the covering and exhibits all the details of construction.[100]

Fig. 32, presented with the discussion by Mr. Thompson, is very interesting. The weather protection mentioned by him can be seen to consist of a kind of flexible shingle applied to each stick of the corbels and cross-ties. The presence of this bridge in far Japan, coupled with the numerous examples of the same type now existing in Northern India, and the Savoy ruin, mentioned in the paper, under "The Ancient Background," seem to prove that the corbelled bridge was known and used across the great Eurasian Continent in the early ages of human development.

Mr. Guppy calls attention to an interesting and true phase of the difference in designing wooden and metal bridges. The comparison might be extended to a somewhat similar difference in fabricating and erecting the two classes. Fig. 23, or its modification, might well be added to Figs. 16 and 28 as examples of the use of metal splices in timber tension members.

The simple method of anchoring a bridge to its abutments described by Mr. Houk, is well worth using on all wooden bridges over streams subject to high water or severe winds. Hundreds of wooden bridges have been floated —pushed by drift or blown by wind—from their bearings, nearly all of which would have escaped serious injury had they been anchored. It seems scarcely necessary to carry the rods to the upper chord. A short rod each side of the lower chord, passing through a yoke block, and each one connecting with two wire lines below, would seem simpler to install than the arrangement described by Mr. Houk.

The combination wood and steel bridges described by Mr. Baker are interesting examples of history repeating itself under new conditions. Mr. Baker's type of construction suggests great possibilities for highway bridges of spans up to 80 ft. Lower chords of channels in one length up to two cars long, with perhaps a plate reinforcement at the center, with angle-blocks and tie-plates riveted on at the shop, or laminated chords of treated plank securely united as one timber with ring dowel, or other connectors, would naturally follow as a type of construction suggested by Mr. Hogue. Preservative treatment is not needed if the ends are protected from dirt accumulations, and if treated wall-blocking is used. Rod verticals and timber upper chords and web diagonals may be used. All spans greater than 45 ft should be housed with roof and side covering, using a roof with a good overhang and a pitch of about 4 in. per ft. Inexpensive trim on facade and sides should be used, and the entire structure painted appropriately to present a bungalow appearance, far surpassing in picturesqueness those of the New England covered type. Shorter spans with lower trusses should have each truss housed, roof and sides. A concrete floor-slab on steel cross floor-beams would prevent the decay complained of, and housing will save the end posts and upper chords. Window openings (see Fig. 13), admit light and increase safety of traffic inside, better the appearance outside, and assist ventilation. The flooring, of whatever character, furnishes adequate lower lateral bracing. The upper chords should be adequately braced in connection with the roof framing.

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[100] An excellent view of the model appears in Mohringer's "Bridges of the Rhine", pub. in 1931.

The writers disagree with Mr. Baker in his statement that wooden bridges are very costly to maintain. It is inferred that the tendency to decay is the cause of his complaint. Piles in sea water, infested with marine insects, open stringers, and plank floors, and, in fact, all timber exposed to the weather, or in contact with masonry or earth, are short-lived in warm climates unless thoroughly treated with a preservative. Truss bridges on the contrary are very durable if protected by housing. (Note the many instances cited in the paper and discussions of bridges a century and more old.) An experience of more than forty years has taught the writers that current maintenance of wooden truss spans is not so costly as maintenance of steel spans. The worst enemy of wooden bridges is fire, due at present largely to cigar and cigarette stubs. Obsolescence and increase of loads, especially in case of railroads, are the usual cause of renewals, rather than physical defects.

Mr. Reed and Messrs. Davis and Jemison have furnished valuable records of early truss bridges west of the Allegheny barrier. These trusses as a whole are based on the Burr design, a worthy example to follow.

The Ramp Creek Bridge built about 1837, with its arches, upper chords, and roof framing of yellow poplar (white wood), and its floor-beams, stringers, and under floor of walnut, causes surprise to an Easterner; but it really shows excellent engineering judgment on the part of its builders and was not less remarkable than the use of the magnificent white pine for bridge work in the East, which timber was once reserved, by law, for masts of the Royal English Navy. The fact that after about 95 years of service the bridge was deemed worthy of re-erection in a location where it would be subject to no heavier traffic than that for which it was built, is certainly worthy of attention.

Failure of floor-beams by longitudinal shear is common if they are overloaded and mortised near their ends for lateral braces, as described for the bridges in Figs. 35 and 37. A well-laid floor is very ample for lateral bracing of a highway bridge. The "bowing" out of line mentioned in the case of the former bridge is quite likely due to flatness of arch and lack of rigidity of the shallow truss with no counters.

The durability of the hand-forged bolts mentioned in the description of this bridge is worthy of note, although not strictly pertinent to a paper on wooden bridges. The more nearly pure an iron is, the better it resists corrosion. Old-time Swedish or Russian iron, or present-day ingot iron, is practically rust proof; while structural steel is not so durable as wood in certain locations.

The Kennedy family, J. J. Daniels, and Lemuel Chenoweth, should be added to the honor roll of names of noted bridge builders.

Another variation of the Burr type of truss was seen in a bridge built by George W. Thayer, of Massachusetts, over the Delaware River, at Narrowsburg, N. Y., in 1859, which had the longest framed trusses (clear span of 262 ft), except Wernwag's "Colossus" (see Fig. 12), of which the writers have knowledge.[101] Each truss had two arches on each side, one above the other, separated by 12 in., or more. A peculiarity in the trusses was horizontal struts ("straining pieces") between the posts, fixed just below the upper, and above the lower, chords where the diagonal braces were framed into the posts. The latter feature was specified in a patent granted to Mr. Thayer in 1845.[102] This and other patents noted in the paper are illustrated and described in a series of articles,[103] by F. B. Brock, Patent Attorney, Washington, D. C., on expired patents for wooden, iron, and combination bridges. The Narrowsburg Bridge was removed when it was about forty years old, while still capable of giving good service (probably for more than twice that period).

The "modern" timber connectors shown by Mr. Landsem are interesting. The multitude and variety of these patented connectors reminds one of the flood of United States patents issued prior to 1880 for bridge trusses of wood, iron, and combinations of the two, each patentee claiming that his arrangement of parts was safer and cheaper than any previously proposed system. The upper chord joint (Fig. 41), seems like a substitute for the American pin-joint. By hooking the four members to a ring in this joint there is no moment induced on the center bolt. Its workmanship must be quite expensive. On this account it is not probable that this joint will be generally used in this country. There is a wide field for their use as smaller connectors in high towers of timber trestles, towers for transmission lines and radio aerials, and especially for portable structures, such as oil-well drilling derricks, observation stands, etc. The old-time rectangular key, with a length of about ten times the depth of dap cut in the timber for it, and two bolts to resist the overturning tendency, is ideal in efficiency, but is perhaps not so economical in these days of machine employment as the circular dap and connector. A simple disk with a central bolt is probably the best solution of the problem.

Mr. Landsem has contributed some valuable facts as to European bridge building, ancient and modern. His statement about the flexibility of laminated arches should not deter builders from their use. If properly secured to stiffening trusses they hold their shape perfectly. Mr. Landsem mentions the materials as being "boards." Arches are built up of 2 to 4-in. "boards" very readily. Radial bolts should be inserted at intervals of not more than 8 ft, and the load applied at points should not be more than 12 ft apart. If greater intervals are allowed buckling may occur.

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[101] *Engineering News*, Vol. 42, p. 122.

[102] *Loc. cit.*, Vol. 9, p. 418.

[103] *Loc. cit.*, Vols. 9 and 10.

Professor Finch contributes a fine historic summary—a splendid frame in which wooden bridges occupy but part of the picture. There is little in it for the practical bridge builder to discuss, but the writers feel disposed to comment upon the feature of statical indetermination and the men named as bringing American practice to rational analysis.

It is a fact that the two all-wooden types best adapted for bridges are inherently statically indeterminate; namely, the Burr (Figs. 14 and 15), and the lattice with supplementary arch (Fig. 17). The elasticity of the material, the tolerance of timber joints to slight distortion, and the natural law of gravitation will bring the stress to the most rigid members. Timothy Palmer's Easton bridge (Fig. 11), was a rather indifferent truss yoked to a flat arch for a lower chord. Theodore Burr improved on this, introducing more indeterminacy but also a large amount of practicality, resulting in wide adoption and approval, as shown throughout this paper and its discussion. The lattice with arch is superior to the Burr type in that it requires much simpler forms of timber, much less skilled labor, and its truss is much better adapted to resist reversals of strain. Still the rank indeterminateness is there, and no one but the schoolmen know or care anything about it. If there is strength enough in both members of the combination acting together, the elasticity and safe "give" at joints will distribute the load exactly according to the static rigidity of each.

Regarding the men named by Professor Finch: Colonel Long was a bridge builder of courage and imagination, although his designs were not so extensively copied as were those of Burr, Town, and Howe. Neither Ithiel Town nor William Howe were builders of bridges. Town was a business man who patented the lattice type which was designed and built in Vermont years before the patent was issued. Sanford Granger, mentioned in the paper, was probably connected with Mr. Town, in securing the patent. Mr. Granger, however, did not invent the type. A lattice bridge was built in 1813 (when Granger was only 17 years old) over Otter Creek, in Pittsford, Vt., many miles from his home. The bridge is still in service after 120 years. It was thought worthy of being floated up river to its place in 1927 when the high water of that year carried it a mile or more from its abutments. Town also patented the timber lattice in 1839. This is also of Vermont birth. William Howe was an architect. In 1839, in connection with Amasa Stone, then 22 years old, he built a bridge of 75-ft span for the City of Springfield, Mass., probably on the Howe principle, because in 1840 he secured a patent, and they built the great bridge over the Connecticut River (Fig. 24), as stated in the paper. The next year Stone bought out Howe's rights in the patent and went into bridge building as a business. Later, he formed a company under the name of Stone and Boomer which engaged in railroad and bridge construction on a large scale.

Concerning the reference to Trajan's Bridge over the Danube River by Mr. Eremin, the figures quoted are those given by Nicholas Bergier[16] and Dion

Cassius.[104] The latter was the Roman historian whose notable history of the Roman Empire in eighty books was written between 217 and 229 A. D.—the century following Trajan's death in 117 A. D.[105] The historian's account of this prodigious and unprecedented work states that it was built to facilitate Trajan's second campaign against a revolted enemy, who killed himself after complete defeat in 106 A. D. The same authority puts the building of the bridge about 104 A. D. The brief existence of the bridge is also noted, for the narrator adds that Trajan's successor, the Emperor Hadrian, caused the arches to be torn down because he was extremely jealous of the fame which this 'wonder of the world"—as the narrator styles it—had brought to his predecessor.[106]

The information on foreign bridges contributed by Messrs. Eremin and Andersen is highly appreciated. The double-web systems of the Gulfoss and other bridges of the Howe type are interesting. They admit the use of smaller diagonals and rods than single systems but, for spans such as those mentioned, it is doubtful if the added work of framing and assembling did not more than balance the saving of material. The omission of side boarding seems wasteful. The lower chord needs protection from the elements more than the upper one. The type of trestle shown was formerly used in some sections in the United States and has the advantage, if properly designed, of allowing the removal of any piece of timber for replacement without interrupting traffic.

Northern Russia and Scandinavia with their wealth of coniferous timber growth are the natural homes of the lattice type of bridge. Baltic fir is well adapted for that type if proper units of stress are used. One-inch round iron or steel drift-bolts, instead of the oak pins shown on Fig. 16, would be advisable for connectors.

The suggestion of Mr. Atwood that early obsolescence may overtake a structure built for all time is of much greater moment than many realize. Most engineers have dug out so much that their predecessors built under the slogan, "build for the future"—good, durable, and expensive work no doubt, but ill adapted to present-day needs—that they should feel that their work will probably go the same way, and that they should build simply what is needed now, and pay for it. The other alternative is to do as has been done many times, to wit, build for the future, paying for it with bonds for coming generations to liquidate long after the constructions have been abandoned or dismantled.

Mr. Meyer presents another specimen of the interesting type of corbelled bridges which have appeared at several points in this paper and its discussions. The great heart of the Eastern Continent lying east of the Ural Mountains and including Western China, in which the Kingdom of Muli is situated, has much to reveal of the early history of Aryan peoples. This type of bridge, called "cantilever" by Mr. Meyer, must be accepted as the prototype of constructed bridges. It is found from the Pacific to the Atlantic Oceans, coming to Europe, apparently, with the Alpine stock of people in very ancient times. Its class had better be called "corbelled," leaving "cantilever" to the distinctive type with the back lever doing duty by spanning an opening.

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[104] Appleton's Cyclopedia, Vol. VI, Article on Dion Cassius, p. 122.

[105] *Loc. cit.*, Vol. XV, Article on Trajan, p. 838.

[106] Bergier's "Histoire, des Grandes Routes de l'Empire Romain," Tome 2, livre 4, p. 750.

Professor Howland submits some appropriate information about the engineering literature available in the Eighteenth and early Nineteenth Centuries. It recalls research made for the paper, the results of which were precluded because of limitations of space. The lack of suitable timber in England, and the resort to cast iron by the English engineers was noted in the paper under "Later European Timber Bridges." For example, this appears by consulting the monumental folio atlas accompanying Rickman's "Life of Telford." That portrays, by fine engravings, 22 stone bridges, including masterpieces such as the famous London bridges; 35 road bridges in the Highlands of Scotland—from arched culverts to those of two or more spans—all of stone; 8 bridges built of cast iron, made in sections and bolted together—two over the Thames and others less known; 2 famous suspension bridges; and only 2 timber bridges of minor importance.

Turning to the European Continent a different story comes to light. The treatises on "The Arts of Construction and Carpentry" by French engineers—Gauthey, Bélidor, Bruyére, Cordier, and especially E'my—are prolific in fine drawings of actual or suggested timber bridges in a great variety of arched and braced contrivances. Perronet, who made his great stone bridges patterns (as it were) for English engineers and the world to follow, developed the designing—as Professor Howland has described—of the heavy timber centers which he required. It is in order to add that the splendid atlas of plates portraying his works has only one drawing of a timber bridge, and that was a "project" for a bridge over the Seine, in Paris, with seven arches of 60-ft span and 10-ft rise, on piers of timber piles. Evidently, he preferred to build for posterity.

Mr. Ellis corroborates others in showing the general prevalence of the Burr type of bridges in certain sections of the country. In some others, particularly in Northwestern New England, the Town lattice prevailed. Burr's designs were not conspicuous for scientific proportioning of parts. His experience and natural flair for heavy construction were probably his only guides. The later changes in character of highways and of loads carried, would find weak spots in many of his bridges, especially those built by others who had not his flair and who built at his type rather than to it. The combination truss and arch, which is the real essence of the Burr patent as finally adjusted, is the absolute final acme of design in wooden bridges of considerable span.

Mr. Ellis adds cumulative evidence to the value of effective covering which is stressed strongly in the main paper.

Mr. Bell's discussion opens an interesting line of conjecture. He states that the bridge at Camp Nelson, near Lexington, Ky., was built by Lewis Wernwag in 1838, and he feels certain that the one at Cheat River (Fig. 35), was constructed by the same builder. In 1855, Nicholas Powers built the

North Blenheim Bridge (Fig. 20), using the same principle of construction—an arch of three rings on the center line of the truss—but he carried his middle truss to the ridge pole of the pitched roof, which brilliant idea, was missed by Wernwag, both at Cheat River and at Camp Nelson. Powers adopted the later type of Long's patent truss which is better adapted for stiffening the arch than the Burr type. The arches of these three bridges follow those of Wernwag's famous "Colossus" in design, but in these later designs he adopted the parallel chord idea of Burr for the stiffening truss instead of framing it on top of the arch (see Fig. 12), thereby giving better support for floor and roof than the original example.

Without doubt the first example of an arch on the center line of a truss was at the Cheat River Bridge, and its author was Lewis Wernwag, who died at Harpers Ferry, Va., in 1843. The two examples show that he adopted, as his later practice, the arch as the main carrier of loads, placed on the center line of the truss designed to hold it from distortion under concentrated loads, even to the extent of the twin arches embedded in a single truss, as at Camp Nelson. His arch segments were hewn to the curve of the arch. This involves the weak factor of timber (longitudinal shear) to a slight extent; and laminated arches laid up with planks 3 to 4 in. thick are now generally used. It remains to examine the truss factor to determine the best type to use for the heavy concentrated loads of the present time both for highway and railroad service. Manifestly, a truss, all the members, chords, and web of which will act in tension or compression, is ideal for the purpose. The Town lattice fills these requirements perfectly, and hence must be given preference. It is gratifying that the idea can be traced back to the illustrious Wernwag.

The sentiment expressed so strongly by Mr. Bell that wooden bridges should not be scrapped because of obsolescence and possibly weak floors, is heartily seconded and upheld by the writers.

Considerable has been written in these discussions about preservative treatment of timber for various purposes connected with bridges. In other publications, treated timber for heavy truss bridges of long span, has been advocated so that roofs and side covering would not be needed. The advisability of the use of preservative to this extent has not been proved either by the discussions or by the experience of the writers. For trestles, small deck spans, and short spans unsuited for covering, treated timber is advisable; but for trusses of spans requiring joints in lower chords, covering is demanded. Esthetic reasons, as well as economy require covering of trussed spans. What looks worse in a landscape than an uncovered wooden through truss bridge especially if it is black with preservative? The bridge shown in Fig. 29, for example, was rightly left uncovered for temporary use only; but its unfinished aspect shows how those designed to be permanent would look.

The general trend of these discussions amply proves the longevity of wooden bridges and justifies their use in localities where they are substantially cheaper than bridges of metal or masonry.

A chronology of the development of the types described in this history and its discussion is given herewith; the first twenty-three types, in this list, built during a period of nearly fifty years, were framed by use of treenails and such small hardware as could be forged by blacksmiths:

Pile-trestle and stringer bridges, crib and stringer bridges after European antecedents	From 1620 forward
Enoch Hale's braced-stringer bridge, at Bellows Falls, Vt.	1785
Timothy Palmer's Essex Merrimac Bridge	1792
Timothy Palmer's Piscataqua and Haverhill Bridges	1794
Timothy Palmer's Georgetown, D. C., Bridge	1796
Timothy Palmer's "Permanent" Bridge, at Philadelphia, Pa	1804–06
Timothy Palmer's Easton, Pa., Bridge	1805–06
Graves' (second) Connecticut River Bridge, at Hanover, N. H.	1796
Windsor, Vt., Bridge, contemporary with the Graves' Bridge	1796
Theodore Burr's Waterford, N. Y., Bridge	1804
Theodore Burr's Trenton, N. J., Bridge	1806
Theodore Burr's Mohawk River Bridge	1808
Theodore Burr's Harrisburg, Pa., Bridge	1816
Lewis Wernwag's "Colossus" at Philadelphia, Pa.	1812
Lewis Wernwag's New Hope, Pa., Bridge	1813–14
Lewis Wernwag's "Economy" Bridge	1810
Earliest lattice-truss bridge of which there is record	1813
Ithiel Town's plank lattice truss, patented	1820
Truss of Stephen H. Long, patented	1830
Ithiel Town's timber lattice truss, patented	1839
Wernwag's Cheat River Bridge, West Virginia	1834
Wernwag's Camp Nelson Bridge, near Lexington, Ky. (Standing in 1933 after 95 years. In both these bridges the arch is on the center line of the truss.)	1838
The Ramp Creek Bridge, Indiana, Burr trusses, built in (Renovated and in service, 1933, after 96 years.)	1837
The Raccoon Creek Bridge, Indiana, Burr trusses. (Still in use, 1933, after 95 years.)	1838
Brunel's experiments in England with preservatives, kyanizing, etc.	1835
Wooden lattice bridges on British railways after Ithiel Town's visit about 1840, before	1846
William Howe's patent for the Howe truss	1840
William Howe's Connecticut River Bridge, at Springfield, Mass.	1840

The Tucker Bridge, at Bellows Falls, Vt., plank lattice	1840
The trusses of Thomas W. and Caleb Pratt, patented	1844
Typical Burr truss railroad bridge (framed with white pine), at White River Junction, Vt.	1848
Howe truss bridge with double arches, at Bellows Falls, Vt.	1850
The unclassified truss of Nicholas Powers, North Blenheim, N. Y.	1855
First bridge across the Mississippi River, five spans, Howe trusses with double arches, at Rock Island, Ill.	1853–56
Second historic bridge at Rock Island, Ill., Howe trusses with curved upper chords and no arches, some time before	1868
The Ledyard Bridge, at Hanover, N. H., timber lattice	1859
Howe truss bridge with double arches (12 spans) at Havre de Grace, Md.	1862–66
The gallows-frame drawbridge	1660
The **A**-frame drawbridge	1835
The jack-knife drawbridge	1845

Within less than a month after publication in "Proceedings" of the closure of the discussion on "A History of the Development of Wooden Bridges," Mr. Snow, joint author of the paper with Professor Fletcher, died suddenly on September 4, 1933, in his eighty-fifth year. His associate in the research can do no less than testify that whatever of authoritative value this paper may possess, is due in very large measure to Mr. Snow's part in its production. Mr. Snow's long experience as Bridge Engineer and, later, as Chief Engineer of the Boston and Maine Railroad—covering the period when wrought iron gradually superseded timber in bridge construction—made him an outstanding authority on the subject. His knowledge of details and his experience in maintenance of such bridges constrained him to adopt a conservative policy in retaining timber bridges as long as they could be made assuredly safe under increasing loadings.

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DISTRIBUTION OF SHEAR IN WELDED CONNECTIONS

By Henry W. Troelsch[1], M. Am. Soc. C. E.

With Discussion by Messrs. F. T. Llewellyn, A. S. Woodle, Jr., Milton Male, William Hovgaard, Charles W. Chassaing, F. E. Fahy, W. H. Jameson, P. Wilhelm Werner, Isamu Ohno, and Henry W. Troelsch.

Synopsis

The purpose of this paper is to analyze mathematically the distribution of shearing stress along the length of longitudinal fillet welds used for connecting structural members. The analysis will be made only for welds that are continuous over the entire length of a lap-joint. In an approximate manner, however, it will apply also to intermittent welds. It can also be used to show the approximate distribution of shear in a line of rivets, provided the deformation or slip of each rivet is assumed to be proportional to its shear.

This paper was prepared in 1929 in an endeavor to interpret theoretically the results of a small number of tests made by the Bureau of Standards in co-operation with the American Bridge Company. Since then several other experimental investigations dealing with the same subject have been undertaken, and it is felt that the mathematical analysis will be of sufficient interest to warrant its publication.

Assume that two members, carrying either tension or compression, each made up of one or more bars of equal section, are spliced by lapping the bars of one member over those of the other in a symmetrical arrangement, and that they are joined by longitudinal fillet welds extending the full length of the lap. Such a splice is shown in Fig. 1.

Note.—Published in November, 1932, *Proceedings*.

[1] Designing Engr., Am. Bridge Co., New York, N. Y.

Fig. 1(*c*) is a diagrammatic representation of this joint, in which, for simplicity in analysis, each member is shown as a single bar. It will be assumed that at any section, *f-f*, the stress in each bar is distributed uniformly over that bar, and that there is no tension or compression in the weld metal. The welds, as well as the metal of the bars, will be assumed to deform according to Hooke's law. The shearing deformation of the weld at any point will then be proportional to the shearing stress per unit of length at that point. This may be expressed by the formula,

$$q = \frac{v}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, q is the shearing deformation of the weld; v, the shear per unit of length of weld; and D, the "detrusion ratio," or the measure of the stiffness of the weld. The value of D depends on the nature of the weld, and must be determined experimentally. It will be measured in the same units as the modulus of elasticity, or unit stress.

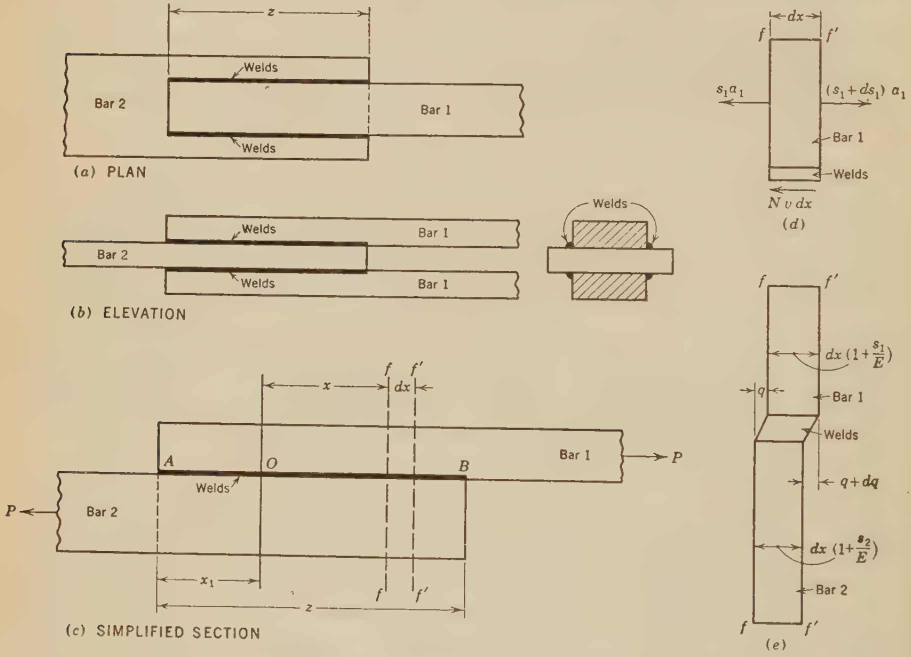
z
Welds
Bar 2
Bar 1
Welds
(a) PLAN
dx
f
f'
s_1a_1
$(s_1+ds_1)\ a_1$
Bar 1
Welds
$N\,v\,dx$
(d)
Welds
Welds
Bar 1
Bar 2
Welds
Bar 1
(b) ELEVATION
f
f'
$dx\,(1+\frac{s_1}{E})$
q
Bar 1
Welds
$q+dq$
$dx\,(1+\frac{s_2}{E})$
Bar 2
(e)
x
dx
A
O
B
Bar 1
P
P
Bar 2
Welds
x_1
z
(c) SIMPLIFIED SECTION

FIG. 1.

The stress in Bar 1 is zero at A and increases to a maximum at B. Similarly, the stress in Bar 2 is zero at B and maximum at A. At some intermediate point, O, the unit stresses in the two bars will be equal and the bars will have equal unit deformations. Proceeding from O toward A the stress in Bar 1, and, consequently, the deformation, becomes constantly less, while in Bar 2 they grow constantly greater. It follows that the displacement between the bars becomes greater as the distance from O increases. Since this displacement is the shearing deformation in the welds, the shear in the welds increases from O to A. Similarly, the shear will increase from O to B. It is evident, therefore, that the minimum shear will occur at O, where the unit stresses in the two bars are equal.

Taking this point, O, as the origin of co-ordinates, let,

a_1 = cross-sectional area of Bar 1.
a_2 = cross-sectional area of Bar 2.
s_1 = unit stress in Bar 1 at any Section *f-f*.
s_2 = unit stress in Bar 2 at Section *f-f*.
E = modulus of elasticity in tension and compression.
N = the number of welds in the joint.
z = the length of the joint.
x_1 = distance of O from the spliced end of Bar 1.
x = abscissa of Section *f-f* with reference to O.
P = total stress (taken as tension) transmitted by the joint.

Consider now that part of the spliced member on one side of Section *f-f*. Since the sum of all horizontal forces must be zero, $s_1 a_1 + s_2 a_2 = P$; whence,

$$s_2 = \frac{P - s_1 a_1}{a_2} \qquad \text{(2)}$$

Take a second section, *f′-f′*, at a distance, dx, from *f-f*, and consider that part of Bar 1 which lies between the two sections (Fig. 1(*d*)). This piece is subject to a tension, $s_1\ a_1$, at Section *f-f*, a tension, $(s_1 + ds_1)\ a_1$, at Section *f′-f′*, and a shear, $N\ v\ dx$, along the welds. From the equilibrium of horizontal forces: $s_1\ a_1 + N\ v\ dx = (s_1 + ds_1)\ a_1$, from which,

$$v = \frac{a_1}{N}\ \frac{ds_1}{dx} \qquad \text{(3)}$$

Substituting this value for v in Equation (1), the shearing deformation is found to be,

$$q = \frac{v}{D} = \frac{a_1}{N\,D}\ \frac{ds_1}{dx} \qquad \text{(4)}$$

Consider now the deformations of those parts of each bar and the welds that lie between Sections *f-f* and *f′-f′*. Under the action of the tensile stress the length, dx, will increase to $dx\left(1 + \frac{s_1}{E}\right)$ for Bar 1, and to $dx\left(1 + \frac{s_2}{E}\right)$ for Bar 2. The increment of the shearing deformation in the distance, dx, will be dq, and the deformation, q, at Section *f-f* will change to $q + dq$ at Section *f′-f′*. These relations are shown in Fig 1(*e*), from which it is evident that $q + dx\left(1 + \frac{s_1}{E}\right) = q + dq + dx\left(1 + \frac{s_2}{E}\right)$; whence,

$$\frac{dq}{dx} = \frac{s_1 - s_2}{E} \qquad \text{(5)}$$

By differentiating Equation (4),

$$\frac{dq}{dx} = \frac{a_1}{N\,D}\,\frac{d^2 s_1}{dx^2} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

Equating these values of $\frac{dq}{dx}$ and substituting for s_2 its value from Equation (2),

$$\frac{d^2 s_1}{dx^2} = \left(\frac{a_1 + a_2}{a_1 a_2}\right)\frac{ND}{E}\,s_1 - \frac{P}{a_1 a_2}\,\frac{ND}{E} \quad\ldots\ldots\ldots\ldots(7)$$

The integral of this is,

$$s_1 = K_1 \sinh\frac{x}{b} + K_2 \cosh\frac{x}{b} + \frac{P}{a_1 + a_2} \quad\ldots\ldots\ldots\ldots(8)$$

in which,

$$b = \sqrt{\frac{a_1 a_2 E}{(a_1 + a_2)\,ND}} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(9)$$

In Equation (8), K_1 and K_2 are two constants of integration, the values of which are found as follows: From Equations (3) and (8):

$$v = \frac{a_1}{N}\,\frac{ds_1}{dx} = \frac{a_1 K_1}{Nb}\cosh\frac{x}{b} + \frac{a_1 K_2}{Nb}\sinh\frac{x}{b} \quad\ldots\ldots\ldots(10)$$

Since the origin, *O*, is at the point of minimum shear, $\frac{dv}{dx} = 0$ when $x = 0$; whence, $\frac{a_1 K_1}{N\,b^2}\sinh 0 + \frac{a_1 K_2}{N\,b^2}\cosh 0 = 0$. As $\sinh 0 = 0$ and $\cosh 0 = 1$, this reduces to $\frac{a_1 K_2}{N\,b^2} = 0$, or $K_2 = 0$. Equation (8) then becomes,

$$s_1 = K_1 \sinh\frac{x}{b} + \frac{P}{a_1 + a_2} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(11)$$

At *A*, where $x = -x_1$, all the stress is carried by Bar 2, and $s_1 = 0$; hence, $0 = K_1 \sinh\frac{-x_1}{b} + \frac{P}{a_1 + a_2}$, or,

$$K_1 = \frac{P}{(a_1 + a_2)\sinh\frac{x_1}{b}} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(12)$$

At *B*, $x = z - x_1$, and all the stress is carried by Bar 1. Hence $s_1 = \frac{P}{a_1}$, and from Equations (11) and (12),

$$\frac{P}{a_1} = \frac{P}{(a_1 + a_2)}\,\frac{\sinh\frac{z - x_1}{b}}{\sinh\frac{x_1}{b}} + \frac{P}{a_1 + a_2} \quad\ldots\ldots\ldots\ldots(13)$$

Expanding $\sinh\frac{z - x_1}{b}\left[= \sinh\left(\frac{z}{b} - \frac{x_1}{b}\right)\right]$, multiplying by $\frac{(a_1 + a_2)}{P}$, and reducing,

$$\tanh \frac{x_1}{b} = \frac{a_1 \sinh \dfrac{z}{b}}{a_2 + a_1 \cosh \dfrac{z}{b}} \quad \text{.....................(14)}$$

which serves to determine x_1 and to locate the origin, O.

From Equations (10) and (12), and from the condition that $\frac{a_1 K_1}{N b^2} = 0$,

$$v = \frac{P a_1}{N\,(a_1 + a_2)\,b} \; \frac{\cosh \dfrac{x}{b}}{\sinh \dfrac{x_1}{b}}$$

Multiplying the numerator and the denominator by b, and substituting for b^2 its value from Equation (9),

$$v = \frac{Pb}{a_2} \; \frac{D}{E} \; \frac{\cosh \dfrac{x}{b}}{\sinh \dfrac{x_1}{b}} \quad \text{.........................(15)}$$

The minimum shear occurs where $x = 0$, and has the value,

$$v_{\text{min.}} = \frac{P b D}{a_2 E \sinh \dfrac{x_1}{b}} \quad \text{.........................(16)}$$

The shear will be maximum at the ends of the joint, that is, where $x = -x_1$ and where $x = z - x_1$. When $x = -x_1$, Equation (15) becomes $v_1 = \dfrac{Pb\,D}{a_2 E \tanh \dfrac{x_1}{b}}$, in which, v_1 is the shear at A. Giving $\tanh \dfrac{x_1}{b}$ its value from Equation (14),

$$v_1 = \frac{P b D}{a_2\,E} \left[\frac{a_2 + a_1 \cosh \dfrac{z}{b}}{a_1 \sinh \dfrac{z}{b}} \right] \quad \text{.................(17)}$$

Similarly, letting $x = z - x_1$, and letting v_2 denote the shear at B,

$$v_2 = \frac{P b D}{a_1\,E} \left[\frac{a_1 + a_2 \cosh \dfrac{z}{b}}{a_2 \sinh \dfrac{z}{b}} \right] \quad \text{.................(18)}$$

From Equations (17) and (18), the maximum shears can be found without finding the value of x_1.

The quantities, $\dfrac{a_2 + a_1 \cosh \dfrac{z}{b}}{a_1 \sinh \dfrac{z}{b}}$ and $\dfrac{a_1 + a_2 \cosh \dfrac{z}{b}}{a_2 \sinh \dfrac{z}{b}}$, in Equations (17) and (18) are always greater than unity, and approach unity as a limit as z increases. Therefore, it is impossible by increasing z to decrease the end shears below the following limiting values:

$$v_1 = \frac{Pb}{a_2} \frac{D}{E} \quad \text{.........................(19)}$$

and,

$$v_2 = \frac{Pb}{a_1} \frac{D}{E} \quad \text{.........................(20)}$$

In ordinary cases the limit is approached rapidly; for example, if a_1 and $a_2 = 4$ and $\dfrac{z}{b} = 12$, $v_1 = 1.0000123$ times the limit.

In making tests to determine the detrusion ratio, D, it is desirable to measure the shearing deformation at the points where the actual value of the shear is equal to the average value over the entire length of the joint. Let x_a be the abscissa of such a point. The average value of the shear is $\dfrac{P}{Nz}$. Then, from Equation (15),

$$\frac{P}{Nz} = \frac{Pb}{a_2} \frac{D}{E} \frac{\cosh \dfrac{x_a}{b}}{\sinh \dfrac{x_1}{b}} \quad \text{.....................(21)}$$

from which,

$$\cosh \frac{x_a}{b} = \frac{E}{D} \frac{a_2}{Nbz} \sinh \frac{x_1}{b} = \frac{b\,(a_1 + a_2)}{a_1\, z} \sinh \frac{x_1}{b} \quad \text{........ (22)}$$

Equation (22) gives two values of x_a which are numerically equal but of opposite sign.

The direct stress at any point in Bar 1 is given by Equations (11) and (12), whence.

$$s_1 = \frac{P}{a_1 + a_2} \left[\frac{\sinh \dfrac{x}{b}}{\sinh \dfrac{x_1}{b}} + 1 \right] \quad \text{................ (23)}$$

Substituting this value of s_1 in Equation (2) and reducing, the stress in Bar 2 is found to be,

$$s_2 = \frac{P}{a_1 + a_2} \left[1 - \frac{a_1 \sinh \dfrac{x}{b}}{a_2 \sinh \dfrac{x_1}{b}} \right] \quad \text{...............(24)}$$

In using these equations it is to be noted that x is negative when measured to the left of the origin.

EXAMPLES

Example 1.—Cross-Sectional Area of Bar 2 Twice That of Bar 1.—Let $a_1 = 4$ sq. in.; $a_2 = 8$ sq. in.; $P = 64$ kips; $N = 4$; $z = 6$ in.; $b = 1.153$; $\frac{z}{b} = 5.21$; $E = 30\,000$ kips per sq. in.; and $D = 15\,000$ kips per sq. in. Fig. 2 shows the results of applying the foregoing equations to this case. By Equations (19) and (20) the limiting values of v_1 and v_2 are found to be 4.61 and 9.22 kips per lin. in., respectively, which are only slightly less than the respective values of 4.73 and 9.30 kips given by Equations (17) and (18).

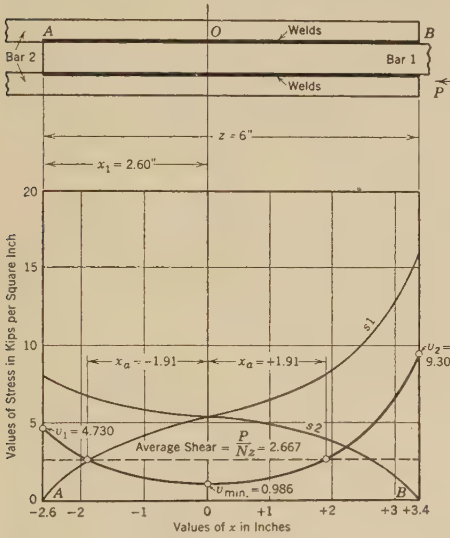

FIG. 2.—CROSS-SECTIONAL AREA OF BAR 2 TWICE THAT OF BAR 1.

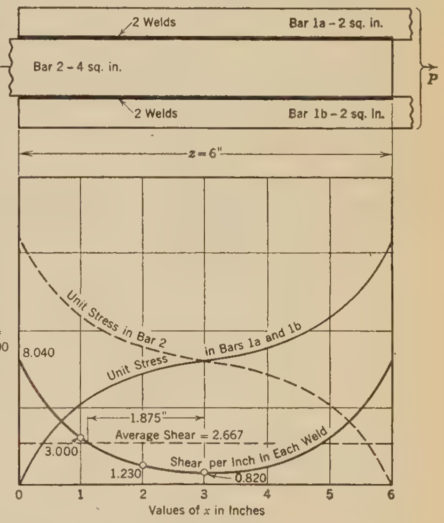

FIG. 3.—CROSS-SECTIONAL AREA OF CONNECTING BARS EQUAL.

Example 2.—Cross-Sectional Areas of Connecting Bars Equal.—Let $a_1 = a_2 = 4$ sq. in.; $z = 6$ in.; $N = 4$; $P = 64$ kips; $b = 1$; $\frac{z}{b} = 6$; $E = 30\,000$ kips per sq. in.; and $D = 15\,000$ kips per sq. in. Fig. 3 shows that, for this case, the end shear is practically equal to the limit of 8.0 kips derived by Equation (19).

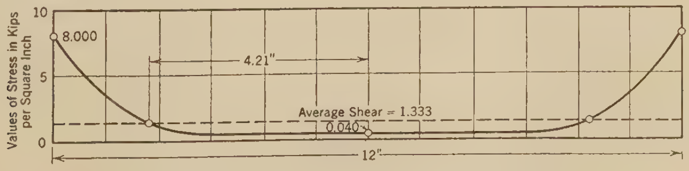

FIG. 4.—EFFECT OF INCREASING LENGTH OF WELD.

Example 3.—Effect of Increasing Length of Weld.—Let the data be the same as given in Example 2, except that $z = 12$ in. and $\frac{z}{b} = 12$. The result is shown by Fig. 4. The difference between v_1 and its limit has become negligible, and it is to be noted that the shear, for a considerable distance on each side of the center, is practically zero. This indicates (1) that the shorter the joint the more nearly uniform will be the distribution of the shear; and (2) that beyond a certain increase in the length of weld no appreciable reduction in the maximum shear will result from further increases in length.

ACKNOWLEDGMENTS

The writer wishes to acknowledge his indebtedness to O. E. Hovey, M. Am. Soc. C. E., and F. T. Llewellyn, M. Am. Soc. C. E., for many valuable suggestions.

DISCUSSION

F. T. LLEWELLYN,[2] M. AM. SOC. C. E. (by letter).—The Engineering Profession is greatly indebted to the author for this valuable contribution toward placing the actual behavior of certain forms of lap-joint on a rational basis. In its major features the analysis now presented is identical with a study which Mr. Troelsch showed to the writer in the summer of 1928, and of which blueprint copies were issued later under date of September, 1928. The writer believes that Mr. Troelsch's method of analysis applies to any form of joint, whether welded, riveted, or otherwise connected, in which the elements that transmit shear are disposed longitudinally. Comment will be given under the heads of mathematics, assumptions, detrusion ratio, and applications.

Mathematics.—Several curious relations, derivable from Mr. Troelsch's expressions, may be pointed out. Regardless of the relation between bar areas, in all cases the shear curve is symmetrical about the origin, although not necessarily of equal extent on both sides. When the area of Bar 1 (Fig. 1) is greater than that of Bar 2 the maximum shear will occur at the end of Bar 1.

Let z_1 be the distance from the origin to the farther end of the joint, and z_2, the distance to the nearer end. Then, for all ratios of bar area and for all values of z, the distance from the origin to the point of average shear lies between z_1 as a maximum, and $\frac{z_2}{\sqrt{3}}$ as a minimum, but never attains either limit.

Some expressions may be simplified by using functions of $\frac{z}{2\,b}$ instead of $\frac{z}{b}$. Thus, let v_1 be the shear at one end of the joint and v_2 the shear at the other end. Then,

$$v_1 + v_2 = \frac{P}{N\,b} \coth \frac{z}{2\,b} \quad \text{.......................} \quad (25)$$

Again, let v_m be the shear at mid-length of the joint; then,

$$v_m = \frac{P}{2\,N\,b} \operatorname{cosech} \frac{z}{2\,b} \quad \text{.......................} \quad (26)$$

In the special case when $a_1 = a_2$, all the author's expressions may be simplified, as follows:

Equation (15) becomes,

$$v = \frac{P}{2\,N\,b} \cosh \frac{x}{b} \operatorname{cosech} \frac{z}{2\,b} \quad \text{..................} \quad (27)$$

Equation (16) becomes,

$$v_{\text{min.}} = \frac{P}{2\,N\,b} \operatorname{cosech} \frac{z}{2\,b} \quad \text{.......................} \quad (28)$$

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[2] Care, U. S. Steel Corporation, New York, N. Y.

Equations (17) and (18) become,

$$v_1 = v_2 = \frac{P}{2\,N\,b} \coth \frac{z}{2\,b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(29)$$

and Equation (22) becomes,

$$\cosh \frac{x_a}{b} = \frac{2\,b}{z} \sinh \frac{z}{2\,b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(30)$$

Assumptions.—The assumptions on which Mr. Troelsch's analysis is based might be acceptable ordinarily. In the present investigation, however, one of them must be qualified in order to represent correctly the actual behavior of joints. Another one must be investigated further if the analysis is to be of practical use.

The assumption that at any section the stress in each bar is distributed uniformly over that bar is not warranted either by the facts as to the bars or by the resulting effect on the welds. A method of determining the variable distribution and of translating it into an equivalent uniform distribution has been presented[3] by W. H. Weiskopf and Milton Male, Assoc. Members, Am. Soc. C. E. Without some such qualification the observed results of certain tests differ from the theoretical values by as much as 30 per cent.

The assumption that there is no tension or compression in the weld metal presumably refers to direct stress lengthwise of the joint. This assumption, although not rigorously correct, is acceptable for the reason that the effect of direct lengthwise stress is apparently negligible as compared with the deformation due to shear.

Mr. Troelsch avoids the necessity of analyzing the internal behavior of the welds by assuming the diagrammatic representation shown in his Fig. 1(*c*), wherein welds of rectangular form are depicted. The internal behavior of such prisms when subjected to shear on opposite sides is readily analyzed. Actually, the welds are of the triangular fillet form indicated in the section to the right hand of the author's Fig. 1(*b*). The internal behavior of a triangular prism subjected to shear on adjacent sides is evidently quite different. Mr. Troelsch makes no attempt to analyze the effect of different forms of weld. He requires that the value of the detrusion ratio, *D*, be determined experimentally for each material and for the cross-sectional dimensions of each form of uniting prism. The need of so doing militates against the predetermination of the manner in which a given joint may be expected to behave. This feature will be discussed in detail later.

Until recently, the assumption that all the elements of the joint deform in accordance with Hooke's law was considered a necessary precedent to analysis. In the discussion as to applications, that will be presented, it will be shown that actual behavior in practice can be reconciled with theory only by assuming that certain portions of the joint do not deform in accordance with Hooke's law. However, the writer is not in position to suggest a practical development of this alternative assumption.

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[3] "Stress Distribution in Side-Welded Joints," by W. H. Weiskopf and Milton Male, Assoc. Members, Am. Soc. C. E., *Journal*, Am. Welding Soc., September and December, 1930.

Detrusion Ratio.—The determination and the proper application of the detrusion ratio, *D,* are essential preliminaries to the establishment of any relation between the theory and the results of tests. Evidently, the detrusion ratio is not the shearing modulus of elasticity of the material in question, but it is related to that modulus by some function of the shape and dimensions of the weld. Mr. Troelsch merely defines detrusion ratio as the measure of the stiffness of the weld. This is very true, but how shall its value be ascertained in practice? In a shearing test within proportional limits on a side-welded joint, if the distribution of stress were constant throughout the length of the weld, and if measurement could be made of the relative motion of the critical elements of the weld, its true detrusion ratio would be determinable by dividing the load (in pounds per square inch) by the amount of this relative motion or detrusion (in inches). The extent to which these conditions are attainable will now be discussed.

It is known by theory and confirmed by numerous tests, that the distribution of stress is not constant throughout the length of the weld. Its variation, however, can be averaged by plotting a sufficient number of observed deformations. As Mr. Troelsch points out, their mean value is the detrusion by which the average shear load is to be divided in order to determine the value of *D.* Thus, the actual condition of variable stress would cause no difficulty if the other condition could be met. Unfortunately, no practicable method has yet been suggested for measuring the relative motion of the critical elements of the weld. All the measurements thus far reported have had to be made between points outside the weld, and, therefore, the value of *D* derived therefrom denotes the stiffness of a zone made up of the weld plus strips of adjacent material rather than the stiffness of the weld itself. This value of *D* may be called the apparent detrusion ratio. No entirely satisfactory method of relating this ratio with the true detrusion ratio of the weld has yet been presented.

If it were physically possible to eliminate the need of establishing this relation by measuring the deformation of points on the weld itself, the question would still remain as to what points should be selected. In the case of a triangular fillet weld the only accessible points on the legs are at the ends of the hypotenuse. Presumably the deformation between these points would be greater than between any other points on the weld, and, therefore, it would not be an indication of the average behavior of the weld which is evidently the behavior contemplated in Mr. Troelsch's definition of *D.*

The most important feature that will pave the way toward the removal of these obstacles, and the reconciliation of the theory with all the observed test results, seems to be a rational analysis of the internal behavior of a triangular prism when subjected to shearing forces on its two adjacent legs, and the translation of this behavior into a quantity which, when multiplied by the known modulus of elasticity in shear, or possibly in tension, of the material, will give the true detrusion ratio, *D,* of the prism. One such analysis is tentatively offered in the paper by Messrs. Weiskopf and Male.[8] As far as the writer is aware, no criticism of this analysis has been presented and no substitute has been offered. A settlement of his feature would be of great value to those interested in the entire problem.

Applications.—All the previous comment has been predicated on the behavior of joints within the proportional limit, in line with one of Mr. Troelsch's fundamental assumptions. Numerous tests have now been made in which care was taken to keep within the elastic limit of the material. In many cases a beautiful correspondence between such tests and Mr. Troelsch's theory has been disclosed. It is proper to inquire what bearing the theory has on actual design, and also to what extent its conclusions are applicable under stresses beyond the elastic limit. The two questions are largely identical for the reason that practically all structures, although designed on the assumption of proportional deformation, involve local concentrations of stress that are far beyond the limits not only of proportionality, but also of elasticity. Engineers have adopted certain conventions which prevent such excess from becoming dangerous. In riveted work, they provide against dangerous concentration of stress on a portion of the rivet shank by restricting the minimum size of rivet that shall be used under specified conditions. It may be that similar legislation is required with respect to the cross-sectional sizes of fillet welds. One fact is quite apparent; namely, that the practical value of analysis such as that presented by Mr. Troelsch is qualitative rather than quantitative. The following example, which illustrates this fact, is interesting because it occurred, not in a laboratory, but in construction of considerable magnitude.

In 1930, during the construction of a new section of subway in New York City, the temporary roadway was carried on 27 and 30-in., rolled-steel girder beams which were spliced, end to end, not at the supports, but at overhanging points. The splices were made by means of side-welded joints. Computations based on Mr. Troelsch's theory indicate that the end shear was five times the average for which the joint was designed. An over-charge of dynamite caused stresses much beyond the contemplated load. If the welds had behaved proportionately they should have failed under their designed load, whereas they did not fail even under the added effect of the blast. The efficiency of the welds as a whole was doubtless due to their relief by a local straining of the ends beyond the elastic limit. This does not indicate any error in the theory. The example is merely one more illustration of the fact that actual construction involves conditions that are not assumed in any single theory.

A. S. Woodle, Jr.,[4] M. Am. Soc. C. E. (by letter).—The fundamental assumption on which this paper is based, is that the shearing deformation in the weld between the bars varies from a minimum at some point in the weld to a maximum at each end. The writer is unable to see the reason for this assumption.

It would seem that like most deformations, that of the bars in question would have a definite direction, and that this direction would be in the line of the stress in each bar. The two bars, therefore, deform in opposite directions from zero at one end of the weld to a maximum at the other end.

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[4] Engr. of Plant, Baldwin Locomotive Works, Philadelphia, Pa.

The increase in unit deformation in each bar would follow a straight-line law, providing only that the bars and the weld between the bars are of uniform cross-section and uniform quality. The unit shear deformation in the weld at any point would be the sum of the unit deformations in the two bars at that point, which are necessarily in opposite directions.

As the locus of the sum of the ordinates of any two straight lines is a straight line also, it follows that the unit shear stress in the weld follows a straight-line law and, in the case of two bars of equal cross-section and the same material, would be a constant throughout the length of the weld.

MILTON MALE,[5] ASSOC. M. AM. SOC. C. E. (by letter).—The results of the analysis on welded joints as reported by Mr. Troelsch are paralleled in the companion paper by Mr. A. Hrennikoff on the distribution of stresses in riveted joints.[5a] The following comments are devoted to Mr. Troelsch's paper.

The "detrusion ratio," D, which is stated to vary with the nature of the weld, and the value of which is to be determined experimentally, may be derived theoretically[6] and its value shown to be a function of the shearing modulus. For the case of a 45° fillet the ratio is equal to $\frac{3}{4}\,G = 8\,700\,000$ lb. per sq. in. In any rational development of a theory of stress distribution in welded joints, the importance of a correct conception of the manner in which the weld acts under shearing deformation warrants further study and experimentation. Indeed, it forms the starting point on which the behavior of the joint as a whole can be predicated.

A graphical illustration of the behavior of fillet welds under the relatively simpler condition of loading—that of an end weld in shear[7]—indicates that the shearing stresses vary along the leg of the fillet by amounts ranging up to 450% of the stress in the bars. It is obvious that for purposes of design some constant intermediate value must be assumed which will approximate the results determined experimentally.

In the analysis by Mr. Troelsch, Equation (15) of the shear curve was derived by assuming that at any transverse section through the bars, the stress carried by those bars was distributed uniformly over their respective total cross-sectional areas, a_1 and a_2. This assumption is one that is commonly made, with a negligible degree of error, in connection with many features of design.

In the case of the welded joints under consideration, however, the error becomes important, because this assumption gives results that vary considerably from many specimens tested. Since Mr. Troelsch developed his equations, many similar equations have been derived independently, but in none but those of Weiskopf,[6] is the deformation in the bars taken into account.

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[5] Technical Asst., U. S. Steel Corporation, New York, N. Y.

[5a] See p. 437.

[6] "Stress Distribution in Side-Welded Joints," by W. H. Weiskopf and Milton Male, Assoc. Members, Am. Soc. C. E., *Journal*, Am. Welding Soc., September, 1930.

[7] "Calcul des Soudures prismatiques à Section Triangulaire," Streletsky and Nikolaieff, First Communications of the New International Assoc. for the Testing of Materials, Group D, 1930.

In this analysis, checked by many measurements made on specimens tested both at Massachusetts Institute of Technology, Cambridge, Mass., and at the University of Pittsburgh, Pittsburgh, Pa., the stresses in the bars are shown to be anything but uniform, and the parts next to the weld are shown to be stressed to a much greater degree than those farther away. Fig. 5 is an exaggerated representation of a side-welded joint under load, in which, Δ is the deformation of the joint bars, and q_0 and q_z are the deformations at the ends of the welds. In the particular case illustrated, the area of the joint bars exceeds the area of the grip bar; therefore, the maximum deformation, q_0, and maximum shear, v_0, occur at the end where the larger area terminates; that is, at the end of the joint bars. To assume that the shear distribution along the weld is the same for both uniform and non-uniform stress in the bars is not correct, and may lead to erroneous results.

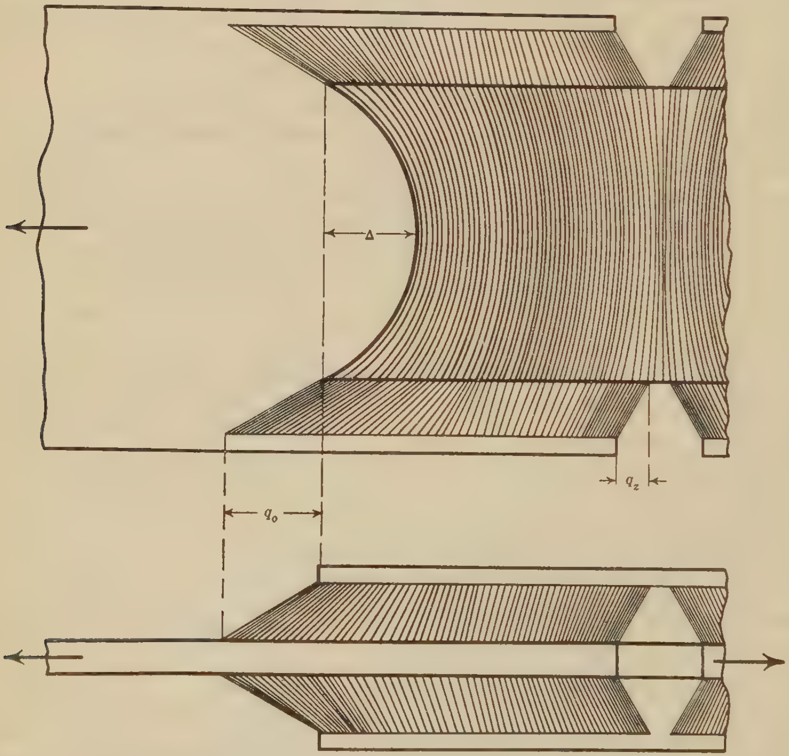

FIG. 5.—EXAGGERATED REPRESENTATION OF A SIDE-WELDED JOINT UNDER LOAD.

It has also been shown[6] that, depending on the ratio of the width of the bar between welds to the length of the weld, the percentage of the total cross-sectional area of the bars that is effective in transmitting stresses into the weld may have some value between 100 and 37, the percentage of effective area decreasing as the bars increase in width. Curves were plotted, which reduced the laborious computations necessary to determine these effective areas to simple slide-rule calculations.

The author's Examples 1, 2, and 3 indicate that in these specific cases the maximum shears are from three to six times the average shear. Since general design practice dictates the use of a safety factor of 4, it would follow necessarily that the welds should fail under loads far less than the working values, and they would so fail, provided these stress ratios are valid beyond the elastic range assumed in the analysis. The satisfactory behavior of welds when tested is evidence that the stresses tend to re-distribute themselves when the elastic limit is exceeded. In other words, the plastic deformation that the welds undergo by the time the ultimate load is reached, produces a shear curve more nearly approaching a horizontal straight line.

What then is the practical value of these analyses? To the writer's mind they serve as a basis for comparing different designs in order to determine which is the more efficient, the results being of a qualitative, rather than a quantitative, nature. As an example, a number of side-welded specimens tested by the Structural Steel Welding Committee of the American Welding Society were analyzed to determine what correspondence existed between their theoretical and actual behavior. The results are given in Table 1. It will be observed that although the correspondence between the computed theoretical behavior and the observed test results does not agree in magnitude, these results do show satisfactory correspondence in trend. It may be concluded, therefore, that although the theory does not enable the designer to predict what the actual ultimate strengths of two or more designs will be, it will permit him to determine in advance which of the two will be the more satisfactory in performance.

TABLE 1.—Comparison Between Theoretical and Actual Behavior of Side-Welded Specimens

Specimen mark	Number of identical specimens	Length of each weld, in inches	Observed average ultimate strength, in kips per linear inch	Theoretical ratio of maximum to average shear	Relative Strength, Percentage	
					Theoretical	Observed
2233...........	8	6	9.5	3.0	100.0	100.0
2243...........	8	8	9.2	3.1	96.8	97.2
2253...........	9	10	8.6	3.4	88.2	90.5
2263...........	8	12	8.7	3.35	89.6	91.6
2433...........	8	6	10.1	2.75	100.0	100.0
2443...........	7	8	9.8	3.1	88.8	96.8
2453...........	8	10	9.8	3.1	88.8	96.8
2463...........	8	12	9.5	3.1	88.8	93.7

William Hovgaard,[8] Esq. (by letter).—The analysis of shearing stresses in longitudinal fillet welds prescribed by the author for double butt-straps gives the same results as those reported by the writer in 1931.[9] The mode in which Mr. Troelsch's equations have been obtained and the form in which they appear, are different, but they are found to give the same results.

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[8] Prof., Naval Constr., Mass. Inst. Tech., Cambridge, Mass.

[9] "The Stress Distribution in Welds"; "The Stress Distribution in Welded Overlapped Joints," *Proceedings*, National Academy of Science, November, 1930; see, also, "The Distribution of Stresses in Welded and Riveted Connections," *Proceedings*, National Academy of Science, June, 1931; and "A New Theory on the Distribution of Shearing Stresses, etc.", *Transactions*, Institution of Naval Architects, London, England, 1931.

The writer has made many experiments and much extended mathematical analysis in an attempt to obtain the physical constants, notably that called, *D,* in the paper, and also to determine the distribution of stresses in both plates.

The problem is of great importance in ships, where longitudinal girders are often attached in relatively short lengths to a continuous surface, such as a deck, subject to a longitudinal strain. The shearing stresses concentrated at the terminals of such girders have often given trouble, as, for instance, at the corners of deck-houses standing on a strength deck, or at the ends of short longitudinals fitted to the bottom plating of a ship. Whether such connections are riveted or welded, the general stress distribution will be the same; excessive stresses are likely to occur at the ends of the girders.

In Civil Engineering the same problem must occur in many cases, as, for instance, in cover-plates on the chords or flanges of girders. The double butt-strap selected for study by Mr. Troelsch is only an individual case among many.

CHARLES W. CHASSAING,[10] ASSOC. M. AM. SOC. C. E. (by letter).—The author has analyzed the shear in welded connections by means of a mathematical theory of elasticity. In so doing he has made some curious assumptions and has obtained some strange results.

For example, his assumption that at any section, *f-f,* the stress in each bar is distributed uniformly over that bar, and that there is no tension or compression in the weld metal, is in error. It leads to the conclusion that the stress in Bar 1 is not the same as that in Bar 2 at their points of contact. The stresses in each of the bars and in the weld metal must be the same at any point common to each.

In deriving his formulas the author has equated the tension strain to the shear strain. According to the mathematical theory of elasticity the cubical dilatation (that is, the change in volume per unit of volume of a small cube of elastic material in strain) is dependent on the tension and compression stresses entirely. The shearing stresses tend only to distort the cube, but not to change its volume.

The formulas give the greatest unit shear at the ends of the weld where it should be zero. It is impossible to have a shearing stress on the surface of any member that is not in contact with a surface. A shearing stress acting lengthwise with the weld must be accompanied by a shearing stress of equal intensity acting perpendicular to it.

According to the formulas the greatest shearing stress in the weld metal is not dependent on the length of the weld. It is unreasonable to suppose, that by increasing the length of the weld, the unit shearing stress would not be decreased.

The author's analysis does not take into account the dimensions of the weld in cross-section. According to his method, the area of the weld in contact with the bars does not enter into the final results.

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[10] Structural Engr., Selden-Breck Constr. Co., St. Louis, Mo.

In the writer's opinion the problem can best be solved by assuming the weld members to act as a beam at the junction of the members with Bar 2. It is assumed that there is no shear stress on a longitudinal section, taken on the center line of Bar 2 as indicated by the dashed line in Fig. 6. Let Fig. 7 represent a half-section of the weld formed by a horizontal plane as shown. The couple, $\frac{P}{4}\left(\frac{t_1}{2} + t_2\right)$, must be balanced, in order to maintain equilibrium, by a couple, Fl. Consider the force, F, to be the resultants of tension and compression stresses distributed over the longitudinal section of Bar 2. It is scarcely necessary to mention that the other half of Bar 2 will balance these stresses.

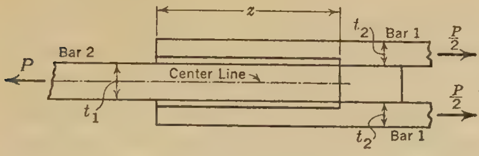

FIG. 6.

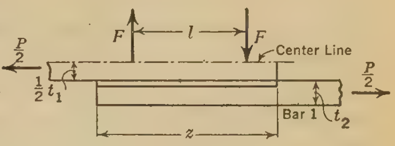

FIG. 7.

There is a moment, $\frac{Pt_1}{8}$, and a shear, $\frac{P}{4}$, acting on each member of the weld. Let w be the width of each member of the weld in contact with Bar 2; z, the length of the weld; and t_1 the thickness of Bar 1. The moment, $\frac{Pt_1}{8}$, and the shear, $\frac{P}{4}$, will have to be resisted by a beam of width, w, and depth, z. Let f_1 be the unit tension or compression stress at the ends of the weld; then,

$$f_1 = \frac{3}{4}\,\frac{Pt_1}{wz^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

The greatest unit shearing stress will occur at the center of the weld and will be one and one-half times the average unit shearing stress. The unit shearing stress at the ends of the weld will be zero. If V_1 is the unit shearing stress at the center of the weld,

$$V_1 = \frac{3}{8}\,\frac{P}{wz} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Referring to the author's Example 1, consider that Bar 2 is $1\frac{1}{4}$ in. by $6\frac{3}{8}$ in. and that $a_2 = 8.96$ sq in. Furthermore, for Bar 1 (1 in. by 4 in.) let $a_1 = 4$ sq in.; $t = 0.5$ in.; and $z = 6$ in. From Equation (31):

$$f_1 = \frac{3 \times 2 \times 64\,000}{4 \times 6^2} = 1\,333 \text{ lb per sq in.}$$

and, from Equation (32):

$$V_1 = \frac{3 \times 2 \times 64\,000}{8 \times 6} = 8\,000 \text{ lb per sq in.}$$

It seems that the tension stress as well as the shearing stress should be taken into account in designing a welded connection.

F. E. Fahy,[11] Assoc. M. Am. Soc. C. E. (by letter).—The author's solution of a rather complicated problem is interesting. It seems worth while, however, to point out that the assumption of a constant value for the detrusion ratio, *D,* will most likely hold only for short joints under low loads and that the assumption of a uniform distribution of tensile or compressive stresses throughout the cross-section of the connected bars will hold only near the centers of long joints, if at all. That the detrusion ratio, *D,* will vary in value throughout the length of the joint will be especially true if the bars are thin and, consequently, incapable of much deformation due to shearing forces acting in a longitudinal direction.

Consider two bars fastened together by a layer of glue, weld metal, or other cementing material, as shown in Fig. 8. Let the thickness of this layer of cementing material be called, *t,* and its shearing modulus of elasticity, E_s. The author's detrusion ratio then becomes equal to $\frac{E_s}{t}$. As *t* decreases, *D* increases, and when *t* is infinitesimally small, *D* approaches infinity; which means that unless there is a definite slipping or yielding of the cement, all the load, *P,* must be transferred from one bar to another within an infinitesimally small distance near one end of the joint. Since this is impossible there will be a plastic yielding of the cement at the ends of the joint and a corresponding adjustment in the distribution of the total shearing stress, *P,* throughout the joint (probably tending toward ultimate uniformity unless there is rupture in the meantime). This assumes, of course, that the bars are drawn up tightly against each other before welding, and neglects the bending effect of eccentricity.

The same argument holds for fillet welds. In Fig. 9 (in which *t* has been exaggerated for clearness), the capacity of the weld to deform, as shown by the dotted lines, *a-a,* depends almost entirely on the distance between the bars, assuming perfect fusion throughout. Under this same assumption of perfect fusion the capacity of the weld to deform, as shown by the dotted lines, *b-b,* depends on the capacity of Bar 1 to deform along those lines, which deformation in the case of a thin bar is quite small.

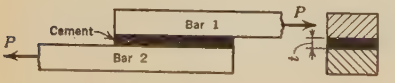

Fig. 8.

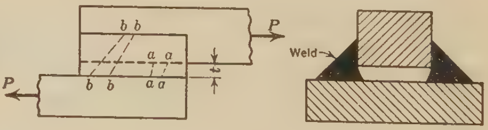

Fig. 9.

Even neglecting the foregoing considerations, however, it will be clear from the author's examples that the shearing stresses at the ends of the joint are beyond the range of elastic deformation of the weld metal. In his

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[11] Asst. Prof., Civ. Eng., Antioch Coll., Yellow Springs, Ohio.

Example 2 the end shear of 8 040 lb per lin in. of weld is equivalent to 21 400 lb per sq in. at the base of a $\frac{3}{8}$-in. fillet, or 30 300 lb per sq in. of throat area; which means that plastic deformation of the weld metal will occur before the extreme values given by Equations (17) and (18) can be reached. A re-distribution of stress throughout the joint will result, with an ultimate distribution curve lying somewhere between the author's curve and the straight line of average shear.

The fact that the author's assumption of a uniform stress distribution throughout the cross-section of the bars is not entirely true, would tend to invalidate the foregoing comments to some extent, although by no means completely. There can be little doubt that even under low loads the high stress concentrations at the ends of the joint will cause plastic deformations that will result in lower stresses near the ends and higher stresses toward the center than the author's results show. This plastic flow near the ends of the joint means (perhaps fortunately) that the strength of a fairly ductile joint is not entirely limited by high shears at the ends.

A complete mathematical solution, which would take into account the deformations of the connected bars due to longitudinal shears, would be valuable; but it is perhaps impracticable. Photo-elastic tests on celluloid models, or a series of carefully made strain-gauge readings at a number of points along actual welded joints, would be of real value in pursuing, experimentally, the line of reasoning that the author has introduced.

W. H. JAMESON,[12] ASSOC. M. AM. SOC. C. E. (by letter).—The formulas derived by the author when used in connection with the results obtained from actual tests reported in 1931 by the American Bureau of Welding[13] show rather surprising results. The author has proved mathematically that lengthening a weld beyond a very nominal length does not increase its strength. If tests proved this to be true, it would be disastrous for the welding industry and, for this reason, the writer feels that the fallacy in the author's assumptions should be shown. The writer has checked the derivation of all the formulas and, if the fundamental assumptions are accepted, has found the actual mathematical work to be correct. Therefore, the error must be in the assumptions since a true mathematical solution must check the results obtained from tests.

Mr. Troelsch states that the stress in Bar 1 (Fig. 1), is zero at A and increases to a maximum at B; and that, similarly, the stress in Bar 2 is zero at B and maximum at A. At some intermediate point, O, therefore, the unit stresses in the two bars will be equal, and the bars will have equal deformation. This assumption implies that the stress at any right section across either Bar 1 or Bar 2 is uniform throughout the bar. To the writer this seems unfounded. The bars are loaded eccentrically with forces along the weld at the edges and the probable deformation of the bars might be represented as shown in Fig. 10.[14]

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[12] Designer, McClintic-Marshall Corporation, Bethlehem Steel Co., Bethlehem, Pa.

[13] Rept., Structural Steel Welding Committee, September, 1931.

[14] *Loc. cit.*, p. 100.

Fig. 1 (*e*) is the elevation of a section of the two bars, which shows actual slipping between the two bars at the weld. Actually, the bars cannot slip at the weld without tearing it so the value on which practically the entire solution is based, is incorrect. At the section, *O*, the stress in the two bars is the same only at the actual point of contact of the two bars and the weld.

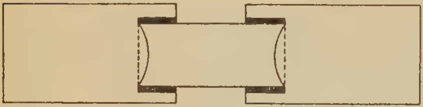

FIG. 10

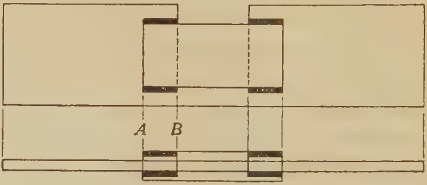

FIG. 11

At any other point on a right section at *O*, the stress is different from that at the point of contact defined herein.

A study of the results of tests given in the report referred to, shows that for a given size of weld, the ultimate stress on the throat of the weld, based on the total effective length[15] averaged about 38 000 lb per sq in. It is somewhat higher (up to 39 500 lb per sq in.) for short welds and somewhat lower (37 500 lb per sq in.) for long welds. This would tend to prove that probably the author's solution has some basis in fact, but since the difference is so small between long welds and short welds, it may be neglected for all practical purposes. For the same specimens, the maximum shear values by Equations (17) and (18) show wide divergence. The results of a few tests are given in Table 2 and all others tested by the writer using the author's formulas show similar results. All these tests were made on specimens with $\frac{3}{8}$-in. fillet welds made up as shown in Fig. 11. In Column (3), Table 2, the total ultimate strengths have been reduced to a common basis of a throat thickness of 0.265 in. before averaging. Columns (4), (5), and (6) show similar values for the average shear, and widely divergent values for the maximum shears by the author's formulas. Since it is unlikely that failure would occur at such different values as the author's formulas would indicate, the writer feels that for welds of ordinary length (certainly up to 12 in. and probably up to

TABLE 2.—ANALYSIS OF TESTS ON WELDED SPECIMENS.

Test No. (1)	Length of weld, z, in inches (2)	Average ultimate strength of weld, p, in pounds (3)	SHEARS, IN POUNDS PER SQUARE INCH		
			Average on the weld, $\frac{P}{0.265z}$ (4)	Maximum, by Equations (17) and (18), Divided by 0.265 $\left(\frac{E}{D}=2\right)$	
				At Section *A*, Fig. 11 (5)	At Section *B*, Fig. 11 (6)
2333.........	7	69 200	37 300	50 300	54 000
2433.........	23	241 000	39 500	95 500	97 500
2443.........	31	319 000	38 800	99 000	114 400
2463.........	47	468 000	37 500	129 300	132 000

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[15] Rept., Structural Steel Welding Committee, September, 1931, p. 45.

at least 24 in.), it is perfectly safe to use the average shearing stress over the entire effective length of the weld. This is the method commonly used and there seems to be no reason to change it, or to fear that welded joints designed by this method will prove to be unsafe.

P. Wilhelm Werner,[16] Assoc. M. Am. Soc. C. E. (by letter).—A rather important problem is treated in this paper. The author has shown that the shear is concentrated near the ends of the weld. It may be of value to know whether there are any means by which a more even distribution of the shear, and thus a better utilization of the length of the weld, could be effected.

Consider the more general case represented by Fig. 12, showing diagrammatically a weld of the same type as that referred to by Mr. Troelsch, but in which the area of the connecting bars decreases uniformly from the full section to a fraction thereof at the ends of the weld. For the sake of simplicity in the following analysis it may be assumed that the two bars are equal, so that the full section is $a_1 = a_2 = a$, and that the area of the end sections is ka, in which, $k < 1$. The author's assumptions as regards the distribution of the direct stress in the bars, etc. (which may be considered to hold true for the type and relative sizes of the bars in question), have been adopted.

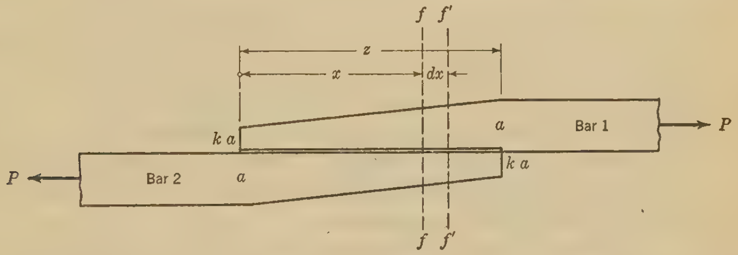

Fig. 12.—Diagrammatical Representation of Weld with Tapering Ends of Bars

The writer has found it more convenient to take the end of Bar 1 as the origin of co-ordinates. Otherwise, using the same nomenclature as the author, the equilibrium condition for that part of Bar 1 on one side of Section *f-f*, may be written, as follows:

$$s_2 = \frac{s_0 - s_1 \left[k + \frac{x}{z} (1 - k) \right]}{1 - \frac{x}{z} (1 - k)} \dots\dots\dots\dots\dots\dots\dots (33)$$

in which, $s_0 = \frac{P}{a}$.

From the equilibrium of horizontal forces on the element between Sections *f-f* and *f′-f′*:

$$v = \frac{a}{z\,N} \left\{ s_1 (1 - k) + \frac{ds_1}{dx} \left[z\,k + x (1 - k) \right] \right\} \dots\dots\dots (34)$$

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[16] Civ. Engr., Stockholm, Sweden.

from which,

$$q = \frac{v}{D} = \frac{a}{z\,N\,D}\left\{s_1\,(1-k) + \frac{ds_1}{dx}\Big[z\,k + x\,(1-k)\Big]\right\} \ldots\ldots (35)$$

and by differentiating,

$$\frac{dq}{dx} = \frac{a}{z\,N\,D}\left\{\Big[zk + x\,(1-k)\Big]\frac{d^2 s_1}{dx^2} + 2\,(1-k)\,\frac{ds_1}{dx}\right\} \ldots\ldots (36)$$

According to the author's Equation (5), combined with Equation (33),

$$\frac{dq}{dx} = \frac{s_1 - s_2}{E} = \frac{z}{E}\,\frac{s_1\,(1+k) - s_0}{z - x\,(1-k)} \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

Equating the values of $\frac{dq}{dx}$ in Equations (36) and (37), substituting $c = \frac{z^2\,N\,D}{a\,E}$, and reducing,

$$\frac{d^2 s_1}{dx^2} + \frac{2\,(1-k)}{z\,k + x\,(1-k)}\,\frac{ds_1}{dx} = \frac{c\,[s_1\,(1+k) - s_0]}{[z\,k + x\,(1-k)]\,[z - x\,(1-k)]} \ldots (38)$$

which corresponds to the author's Equation (7), with $a_1 = a_2 = a$.

As far as the writer knows, it is not possible to integrate Equation (38) directly. A numerical solution is obtained, however, by breaking up the equation into two simultaneous differential equations of the first order. This will be illustrated by the following examples.

Example 1.—Letting $k = \frac{1}{2}$, and assuming the same numerical values as in the author's Example 2, $c = \frac{6^2 \times 4 \times 15\,000}{4 \times 30\,000} = 18$ in., and $s_0 = \frac{64}{4} = 16$ kips per sq in. After reducing, Equation (38) becomes,

$$\frac{d^2 s_1}{dx^2} + \frac{2}{6+x}\,\frac{ds_1}{dx} = \frac{108\left(s_1 - \frac{2}{3}\,s_0\right)}{(6+x)\,(12-x)} \ldots\ldots\ldots\ldots\ldots (39)$$

In Equation (39), substituting $\frac{ds_1}{dx} = w$ and $\frac{d^2 s_1}{dx^2} = \frac{dw}{dx}$:

$$\frac{dw}{dx} = \frac{108\left(s_1 - \frac{2}{3}\,s_0\right)}{(6+x)\,(12-x)} - \frac{2\,w}{6+x} \ldots\ldots\ldots\ldots\ldots (40a)$$

and,

$$\frac{ds_1}{dx} = w \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40b)$$

Equations (40*a*) and (40*b*) are two simultaneous differential equations of the first order, which can be solved by the numerical integration method developed by Runge.[17] To save space, the somewhat elaborate numerical calculations will be omitted here, and only the result will be given. It may be mentioned that the values of both s_1 and $w = \frac{ds_1}{dx}$ are obtained simultaneously in the same integrating operation. The values of s_2 and v are then calculated according to Equations (33) and (34), respectively, with $k = \frac{1}{2}$.

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[17] "Ueber die numerische Auflösung von Differentialgleichungen", von C. Runge, *Mathematischen Annalen*, Bd. 46 (1895), or "Leitfaden sum graphischen Rechnen", von R. Mehmke, Leipzig, 1917.

The result is shown graphically in Fig. 13. It is seen that the maximum shear at the ends of the weld has decreased only a comparatively moderate amount, namely, from 8.04 to about 7.5 kips per sq in.

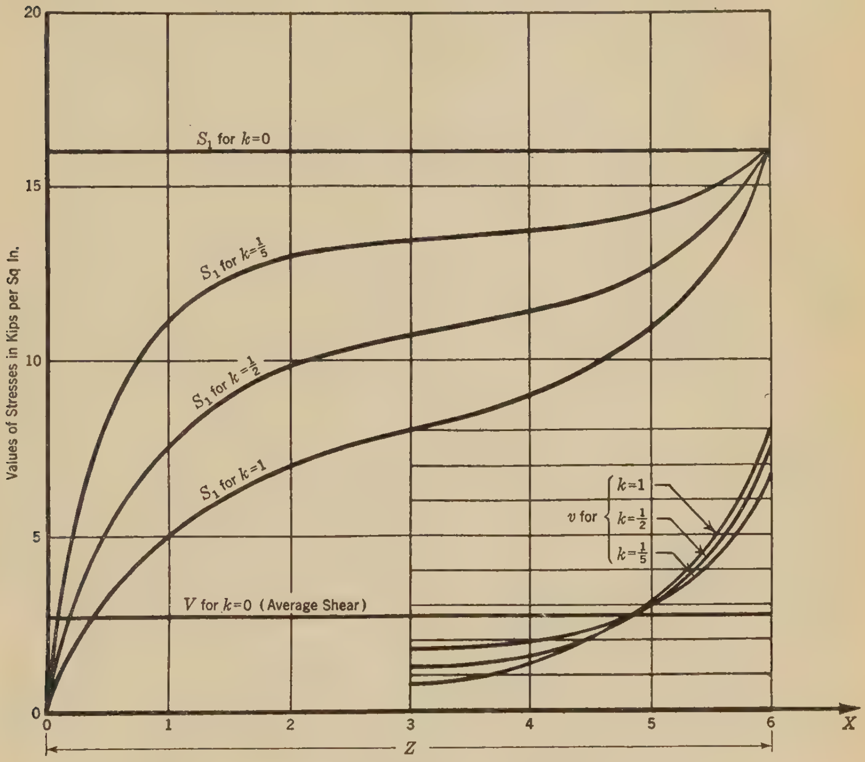

FIG. 13.—STRESSES IN WELD WITH TAPERING ENDS OF BARS

As the shear is symmetrical about the center line of the weld, it has been plotted for only one-half the length of the weld. The unit stress, s_2, is identical to s_1, but reversed, and, therefore, has not been given in Fig. 13.

Example 2.—Let $k = \frac{1}{5}$, and the other values as given previously. Equation (38) then becomes:

$$\frac{d^2 s_1}{dx^2} + \frac{4}{3 + 2x}\,\frac{ds_1}{dx} = \frac{135\left(s_1 - \frac{5}{6}\,s_0\right)}{(3 + 2x)\,(15 - 2x)} \quad\ldots\ldots\ldots\ldots(41)$$

from which,

$$\frac{dw}{dx} = \frac{135\left(s_1 - \frac{5}{6}\,s_0\right)}{(3 + 2x)\,(15 - 2x)} - \frac{4\,w}{3 + 2x} \quad \ldots\ldots\ldots\ldots\ldots (42a)$$

and,

$$\frac{ds_1}{dx} = w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (42b)$$

The integrals of Equations (42*a*) and (42*b*) are obtained in the same manner as for Equations (40*a*) and (40*b*). The values of s_2 and v are then calculated according to Equations (33) and (34), respectively, with $k = \frac{1}{5}$.

This result is shown also graphically in Fig. 13. It is seen that in this case the maximum shear at the ends of the weld has further decreased, that is, to about 6.5 kips per sq in. This decrease is considerable when compared with the average shear, which is $v = 2.667$ kips per sq in.

For comparison, the values of s_1, s_2, and v from the author's case, Example 2, have also been plotted in Fig. 13. It appears that the more the bars are tapered, the more evenly will the shearing stresses be distributed over the length of the weld. If the ends of the bars were wedge-shaped ($k = 0$), the shear would be constant throughout the length of the weld, and would be equal to the average shear. The unit stresses in the bars would then likewise be constant throughout the length of the weld, and would be equal to s_0.

Even a short taper of the very ends of the bars, reaching over only part of the weld, would be likely to effect a considerable decrease of the shear at the ends of the weld. A reduction of the shearing stresses in these places is desirable not only because it effects a more favorable distribution of the stresses, but possibly also because a concentration of the stresses at points of discontinuation, such as those constituted by the ends of the weld, may encourage the beginning of failure. As far as the writer is aware, a taper reaching at least over part of the weld at the very ends of the bars should not generally meet with any practical difficulties.

ISAMU OHNO,[18] ESQ. (by letter).—In connection with the paper by Mr. Troelsch the writer wishes to call attention to the application of the method to a plate to which a shorter reinforcing plate has been welded. Let Fig. 14 be a representative part of such a reinforced plate, for which the center of co-ordinates is at Point O (Fig. 14(*b*)). Since the reinforcing plate is sym-

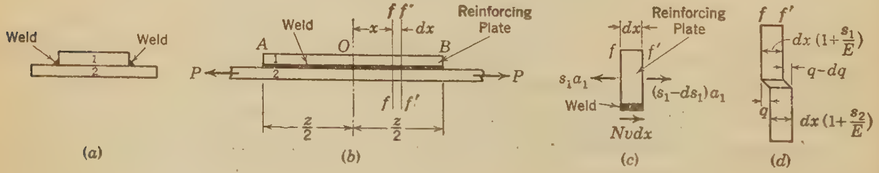

FIG. 14

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[18] Tokushima, Japan.

metrical, only the right half, $O\,B$, need be considered. Using the author's notation and logic, his formulas, Equations (2), (8), (9), and (10) are derived as in the original paper.

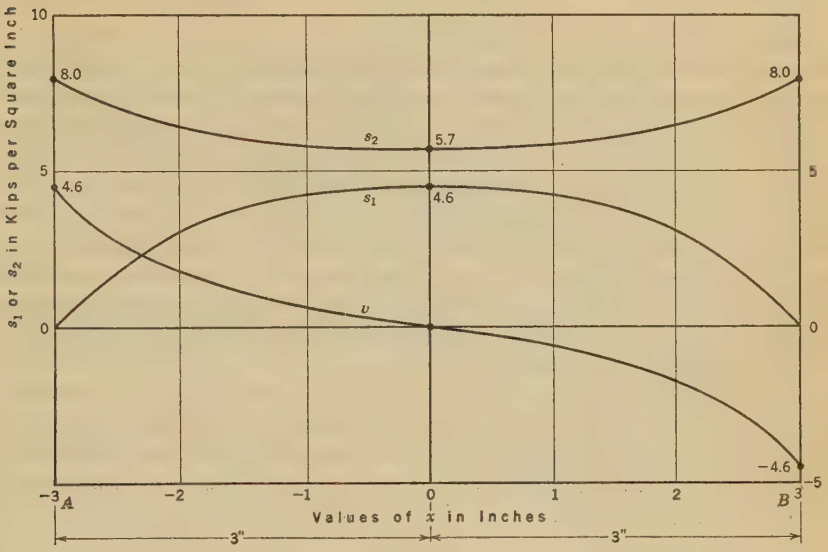

FIG. 15

In the present case, however, the boundary conditions are different; so that $v = 0$ when $x = 0$, and $s_1 = 0$ when $x = \frac{z}{2}$. Therefore, $K_1 = 0$ and

$$K_2 = \frac{P}{\cosh \frac{z}{2\,b}\,(a_1 + a_2)}.$$

Substituting these values in the author's formulas his Equations (23), (24), and (15) may be rewritten, as follows:

$$s_1 = \frac{P}{a_1 + a_2}\left[1 - \frac{\cosh \frac{x}{b}}{\cosh \frac{z}{2\,b}}\right] \quad \text{.....................(43)}$$

$$s_2 = \frac{P}{a_1 + a_2}\left[1 + \frac{a_1 \cosh \frac{x}{b}}{a_2 \cosh \frac{z}{2\,b}}\right] \quad \text{...............(44)}$$

and,

$$v = -\frac{P\,b\,D \sinh \frac{x}{b}}{a_2\,E \cosh \frac{z}{2\,b}} \quad \text{......................(45)}$$

If the area, a_1, of the reinforcing plate is 4 sq in., the area, a_2, of the main plate is 8 sq in., and if the other quantities are the same as in the author's Example 1, the resulting shears and tensile stresses will be as shown in Fig. 15.

HENRY W. TROELSCH,[19] M. AM. SOC. C. E. (by letter).—In preparing the analysis presented in this paper, the writer purposely made the underlying assumptions as simple as possible. Any other procedure would have involved complicated mathematical expressions, which would have tended to obscure the results. However, for stresses within the elastic limit, the theory offers a picture of the distribution of shear in longitudinal fillet welds which is qualitatively correct. Whether it can be made to yield numerically correct values of the shear in various parts of the welds can be determined only by checking against the results of tests. Such tests as have come to the writer's attention indicate that the distribution of shear can be closely represented by Equation (15), provided the proper values of the detrusion ratio, D, and the areas, a_1 and a_2, are used.

The closeness of the correspondence between test and theory is shown in Fig. 16, plotted from the results of two tests selected from those made by the late J. Hammond Smith, M. Am. Soc. C. E., at the University of Pittsburgh, Pittsburgh, Pa.[20] The plotted points show the measured slip between gauge points on the bars. In Fig. 16(b) the curve is plotted from the equation,

$$q = \frac{v}{D} = \frac{Pb}{a_2\,E}\;\frac{\cosh \dfrac{x}{b}}{\sinh \dfrac{x_1}{b}}$$

using for a_1 and a_2 the actual area of the bars, and for D the value, 7 100 000, which is required to make the curve fit the plotted points.

In Fig. 16 (a), the plates are wider, and the effect of the non-uniform distribution of their tension, to which Mr. Llewellyn and Mr. Male have referred, becomes evident. To make the formula agree with the test results it is necessary to reduce the values of a_1 and a_2 from the actual area, 3.0, to 1.59 for a_1 and 2.44 for a_2. The detrusion ratio, D, is 5 480 000. The amount of reduction in the areas of the plates is less than that obtained by Messrs. Weiskopf and Male.[3] It is to be noted, however, that the results agree with the findings of Messrs. Weiskopf and Male to the extent that the reduction for Bar 1 is greater than that for Bar 2.

The detrusion ratio obtained from these tests is not that for the welds alone, but for the welds plus adjacent parts of the bars. For purposes of analysis it is impossible to say where the weld stops and the plate begins. It may be that an analysis, which, like the foregoing, is based on "equivalent areas" and a slip between points near the welds, is as near a practical solution as can be expected.

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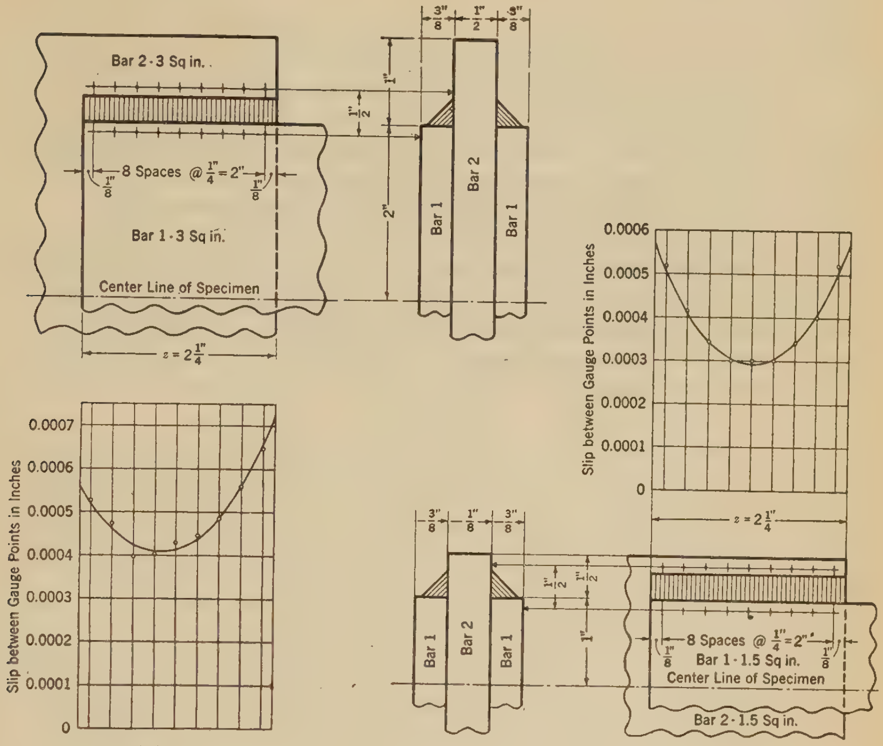
[19] Designing Engr., Am. Bridge Co., New York, N. Y.

[20] "Stress-Strain Characteristics of Welded Joints," by J. Hammond Smith, *Journal*, Am. Welding Soc., September, 1929, p. 79.

The fact that the theory, with such modifications as have been suggested, shows close agreement with measured deformations, indicates that it is substantially correct. The theories suggested by Mr. Woodle and Professor Chassaing, based on a different set of fundamental assumptions, show no such agreement with the tests.


Bar 2 - 3 Sq in.
8 Spaces @ $\frac{1}{4}''=2''$
$\frac{1}{8}''$
$\frac{1}{8}''$
Bar 1 - 3 Sq in.
Center Line of Specimen
$z = 2\frac{1}{4}''$
$\frac{3}{8}''$
$\frac{1}{2}''$
$\frac{3}{8}''$
1''
$\frac{1}{2}''$
2''
Bar 1
Bar 2
Bar 1
Slip between Gauge Points in Inches
0.0007
0.0006
0.0005
0.0004
0.0003
0.0002
0.0001
0
$\frac{3}{8}''$
$\frac{1}{8}''$
$\frac{3}{8}''$
$\frac{1}{2}''$
$\frac{1}{2}''$
1''
8 Spaces @ $\frac{1}{4}''=2''$
Bar 1 - 1.5 Sq in.
Center Line of Specimen
Bar 2 - 1.5 Sq in.

(*a*) (*b*)

FIG. 16.

It is to be observed that in both these tests the unit shears for which the deformations were measured, are such as would be applied to similar welds in designing practice. The average shear in the welds is 2 670 lb per lin in. of $\frac{5}{16}$-in. fillet weld.

It is undoubtedly true, as Mr. Llewellyn and Mr. Male have pointed out, that welded joints will carry, without failure, loads which according to this theory would stress the ends of the welds beyond the breaking point. This can be explained by assuming that after a weld has been stressed to the yield point, further deformation occurs without much increase in stress, so that for a considerable distance near each end of the joint the welds are stressed to the yield point or slightly beyond it. A joint in such a state of stress will probably carry its load satisfactorily if the load is constant. On the other hand, as has been pointed out[21] by H. M. Priest, M. Am. Soc. C. E., impact or variable loads, and especially loads which alternate between tension and compression, might lead to a fatigue failure because of repeated over-stressing of the ends of the welds. Inasmuch as variable and alternating stresses occur regularly in bridge trusses, an experimental investigation of the behavior of welded joints under the action of such stresses should yield valuable information.

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[21] "The Practical Design of Welded Steel Structures," by H. M. Priest, *Journal*, Am. Welding Soc., August, 1933.

Mr. Werner's analysis of a joint with tapered bars indicates that to obtain anything approaching a uniform distribution of shear, the bars must be tapered nearly to a knife-edge; that is, k must be nearly zero; for in Mr. Werner's Example 2, with k as small as one-fifth, the end shear is still 2.5 times the average shear. The fact that a taper tends to eliminate excessive local stresses at an abrupt change in section has long been recognized in the practice of finishing re-entrant angles of machined parts with a fillet.

Mr. Ohno's application of the theory to a reinforcing plate is along the line suggested by Professor Hovgaard in connection with longitudinal girders or deck-houses welded to the deck of a ship. It should be possible to make a similar application to the cover-plates of a plate girder, although the mathematical analysis will be complicated by the variation of the flange stress along the length of the girder.

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WORK OF RIVETS IN RIVETED JOINTS

By A. Hrennikoff,[1] Esq.

With Discussion by Messrs. Henry W. Troelsch, Henry B. Seaman, A. H. Finlay, F. P. Shearwood, W. P. Roop, H. N. Hill and Marshall Holt, Donald E. Larson, A. E. R. de Jonge, and A. Hrennikoff.

Synopsis

An investigation of how a force acting on a joint, made up of plates, divides itself among the rivets, is the subject of this paper. Part I is a qualitative study, with some observations on conventional design methods. This is followed, in Part II, by the derivation of formulas for the forces developed by the rivets in a few simple types of joints, based on the assumption that a rivet develops a force proportional to deformation. In Part III the formulas in Part II are applied to the determination of the numerical values of the forces of the rivets, based on the value of the coefficients found analytically.

PART I.—Conventional Design and Actual Work of Riveted Joints

Despite their widespread use in engineering, riveted joints and the actual work in such joints when stressed, are not well understood. A rigid theoretical investigation of this subject is difficult and, in many respects, indefinite. It requires certain assumptions to start with, which may be not quite true for all cases. For example, conditions under which a rivet has been driven would affect its work considerably. A badly driven field rivet would act quite differently from a good shop rivet.

One difficulty is that inelastic deformations begin to take place a considerable time before the limit of safety is reached. Furthermore, formulas that could be developed along more or less rational lines would be difficult to use in actual practice, except in plate work (boilers, tanks, etc.). Nevertheless, the writer attempts to present herein an analysis which, while admittedly not perfect, may raise some questions of interest for discussion.

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"Standard practice" has developed several arbitrary methods of design that remain in vogue mostly because they are convenient. Consider the simple case of a lap-joint (Fig. 1) with two plates in tension, connected by several rows of rivets equally spaced. Imagine the joint to be divided longitudinally into equal strips with one row of rivets in each, and consider one strip as representative of the work done by the entire joint.

Let $w =$ the width of the strip; t, the thickness of each plate; d, the diameter of the rivet; p, the pitch of rivets; n, the number of rivets in each longitudinal strip; and F, the total pull on one strip.

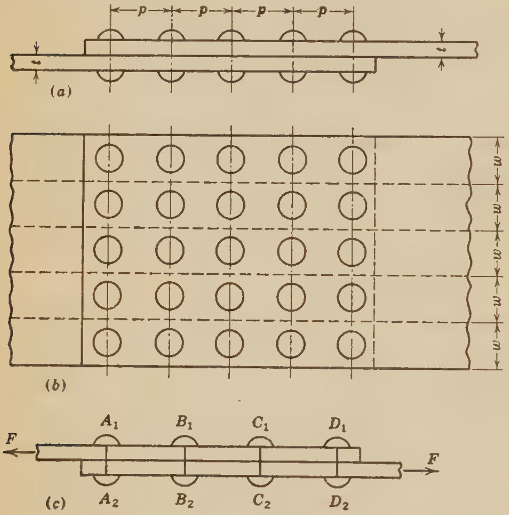

FIG. 1.

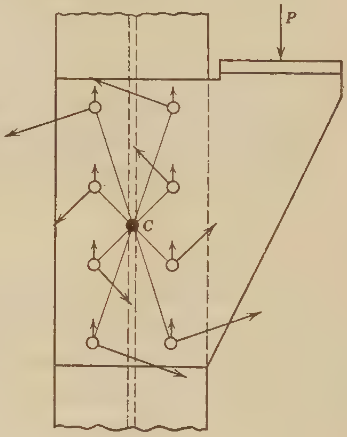

FIG. 2.

It is customary to assume that the force, F, acting on a joint, is divided equally among the rivets, so that each rivet is stressed by the force, $\frac{F}{n}$, in shear and in crushing. The unit stresses are expressed by the following equations:

For shear,

$$s_s = \frac{F}{n \frac{\pi d^2}{4}} = \frac{4F}{\pi d^2 n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

For crushing,

$$s_c = \frac{F}{t d n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Assuming certain safe unit stresses in shear and crushing, s_s and s_c, the number of rivets required to transmit the force, F, safely must be not less than that computed by either of the following equations:

$$n_s = \frac{4F}{\pi s_s d^2} = k_s \frac{F}{d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and,

$$n_c = \frac{F}{s_c\, t\, d} = k_c \frac{F}{t\, d} \quad \text{.......................(4)}$$

in which, k_s and k_c, depending on s_s and s_c, are established by experiment.

It must be pointed out that a rivet cannot work in shear and bearing without deforming; therefore, the standard accepted theory is based on the assumption that a rivet is deformed even when it is under a small stress. Although this may be true in the case of imperfect rivets, a perfect one does not deform until the friction force between the plates, caused by tension in the shank, is exceeded.

Sometimes engineers state that a rivet is good only as long as it does not deform at all and that it should be designed on the basis of friction only. A safe force for one rivet on this basis would be also proportional to the cross-sectional area of the rivet, thus:

$$F_1 = cd^2 \quad \text{.........................(5)}$$

in which, c is a coefficient, depending on the coefficient of friction between the plates, temperature, and other conditions of driving the rivet, Young's modulus, and the coefficient of expansion of the metal For the total number of rivets in a row this would give an expression similar to Equation (3) in the design for shear, namely,

$$n_f = k_f \frac{F}{d^2} \quad \text{.........................(6)}$$

in which, k_f again would be established by experiment.

It must not be concluded, however, that design for shear is the same as that for friction. This simple method of designing on the basis of friction seems to be quite correct in the case of only one rivet in a longitudinal row, or two rivets in case of plates of equal thicknesses. For more than two rivets in a row the method breaks down, as may be seen if deformation of the plates is taken into account.

Let Fig. 1(c) represent a lap-joint of the type shown in Fig. 1(a) with four rivets in a row. If the rivets are working in friction only the plates must not slip with respect to each other under the action of Force F; that is, the lengths, $A_1 B_1$, of the upper plate, and $A_2 B_2$, of the lower plate, which are equal before the force begins to act, must remain so during the action of force. In other words, the elongation of the section, $A_1 B_1$, in the upper plate under the force, F, must equal that of the corresponding section, $A_2 B_2$, in the lower plate. That means that the tensile forces in each plate must be equal (the plates being of the same thickness, width, and metal), which can happen only if Rivet A has transmitted to the lower plate one-half the force, F, that made the tensile forces equal in both plates in the section, A-B.

If a larger or a smaller force is transmitted by the rivet, A, the plates do not stretch equally, they slip, and at least one of the rivets deforms. Similarly, if the rivet, C, does not deform, the parts, B-C, of the upper and lower plates must stretch equally, which is possible only if the rivet, B, does no work. The same reasoning would show that Rivet C does no work either and that Rivet D transmits the other half of the force, F; that is, if the rivets work in friction only, it is really the outer rivets, one on each side of the joint, that actually work, the intermediate rivets performing no work. Hence, it follows that a method of design based on allowing a uniform value for each rivet, working in friction, is wrong.

What probably happens in the row of rivets, when the strip (Fig. 1), is subjected to a gradually increasing force, F, is that when F is small, only the outer rivets, A and D, do any work, each transmitting by friction a force, $\frac{F}{2}$. As soon as the increasing tensile force, F, equals the maximum friction force that can be developed by two rivets, the outer rivets begin to deform as the plates slip at the points, A and D.

Then the length, $A_1 B_1$, is greater than $A_2 B_2$, since $A_1 B_1$ is elongated by the greater force. This brings the rivet, B, into play in such a way that it takes up the remainder left by the rivet, A; that is $X_B = \frac{F}{2} - X_A$, in which, X_A and X_B are the forces developed by the rivets, A and B.

Force X_B is developed entirely by friction, and X_A is the sum of the forces developed by friction and shear, and crushing ($X_A = X_{Af} + X_{As}$). The force, X_{Af}, developed by friction at Rivet A probably remains more or less constant, as the tensile force continues to increase, and it is equal or (more likely) somewhat less than the maximum friction force that a rivet can develop before it begins to deform.

It is reasonable to assume that the other part, X_{As}, of the total force, X_A, developed by the rivet (that due to shear), is proportional to the slip of plates at Point A.

In the foregoing example, Rivet C works the same as Rivet B, and Rivet D works the same as Rivet A. As the tensile force, F, increases still more both X_A and X_B increase; at a certain time, the rivet, B, cannot develop its share by friction alone and it also begins to deform. The force, X_B, taken by Rivet B after that is the sum of the forces developed by friction and shear, and crushing ($X_B = X_{Bf} + X_{Bs}$). The force developed by friction, X_{Bf}, is the same as that of Rivet A. When X_B increases, the friction force, X_{Bf}, remains constant and the force developed by shear and crushing, X_{Bs}, increases in proportion to the deformation.

If the friction force, X_{Af}, and the coefficient of proportionality between the shearing force developed by a rivet and its deformation are determined by experiment, a reasonably rigid theory of a riveted joint may be deduced by means of a mathematical analysis.

PART II.—FORMULAS FOR THE FORCES ACTING ON RIVETS

Following the logic of Part I, it is evident that the value of X_{Af} may vary widely for the same size of rivet, depending on how well the rivet has been driven. For a bad rivet it may approach zero. In that case, the rivet works only in shear and crushing and develops a force proportional to deformation. Formulas developed under this assumption will evidently be on the safe side. The assumption is in agreement with general practice in the design of riveted joints that transmit bending moments, as in the case of a joint in the web of a plate girder, or a bracket connection to a column (Fig. 2).

The usual procedure in these cases is to find the center of gravity of the rivets (Point *C*, Fig. 2) and to allow for each rivet a force, proportional to the distance from the center of gravity, and at right angles to the line from the center of gravity to the rivet. Equating the sum of the moments of all the rivets to the external moment, the force due to bending can be found for each rivet.

This procedure is really based on two assumptions: First, that the ends of the connected parts, considering bending moment only, turn with respect to each other about the center of the rivets, due to which fact the rivets become deformed proportionally to their distances from the center of gravity of rivets; and, second, that the forces of the rivets are proportional to their deformations.

This second assumption will be adopted in the following examples for the case of a lap-joint (or butt-joint) of plates in direct tension or compression.

Examples

Example 1.—Lap-Joint with Three Rivets.—Let Fig. 3 represent a longitudinal strip of a lap-joint similar to Fig. 1(*c*), held by three rivets, *A*, *B*, and *C*. It is evident from symmetry that Rivet *A* develops the same force, *X*, as Rivet *C*; then the middle rivet, *B*, takes the remainder, or $F - 2X_1$.

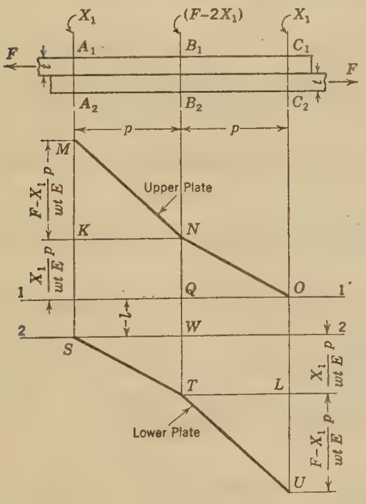
X_1
$(F-2X_1)$
X_1
F
A_1
B_1
C_1
F
t
t
A_2
B_2
C_2
p
p
M
$\frac{F-X_1}{wtE}p$
Upper Plate
K
N
$\frac{X_1}{wtE}p$
1
Q
O
1
l
2
W
2
S
T
L
$\frac{X_1}{wtE}p$
Lower Plate
$\frac{F-X_1}{wtE}p$
U

FIG. 3.

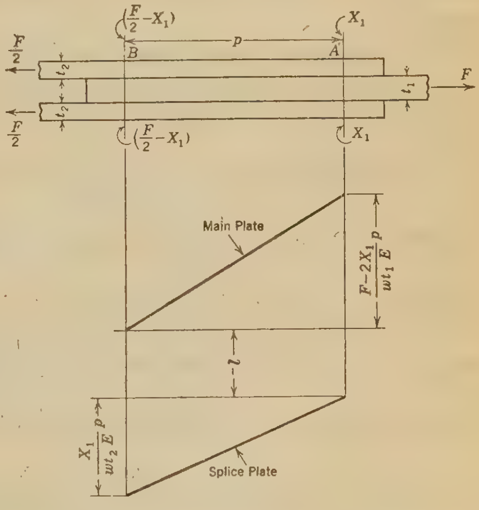
$(\frac{F}{2}-X_1)$
X_1
p
$\frac{F}{2}$
B
A
t_2
t_1
F
t_2
$\frac{F}{2}$
$(\frac{F}{2}-X_1)$
X_1
Main Plate
$\frac{F-2X_1}{wt_1E}p$
l
$\frac{X_1}{wt_2E}p$
Splice Plate

FIG. 4.

In the upper plate a tensile force, $F - X_1$, acts between the rivets, *A* and *B*, and a force, X_1, acts between the rivets, *B* and *C*. In the lower plate a tensile force, X_1, acts between A_2 and B_2 and the force, $F - X_1$, acts between B_2 and C_2.

Under these tensile forces the plates elongate. The portions, $A_1 B_1$, and $B_2 C_2$, each stretch an amount equal to $\frac{F - X_1}{w\,t\,E}\,p$, and the portions, $B_1 C_1$ and $A_2 B_2$, stretch an amount equal to $\frac{X_1}{w\,t\,E}\,p$; or, in other words, the points, A_1 and B_1, move leftward from Point C_1, and the points, B_2 and C_2, move rightward relative to Point A_2.

Curves of displacements of different points on the plates with respect to Point C_1 on the upper plate and Point A_2 on the lower plate, are shown in Fig. 3. The ordinates represent the relative movements of points on the plates having corresponding abscissas, referred to C_1 or A_2; thus, $N\,Q = W\,T = \frac{X_1}{w\,t\,E}\,p$; and, $M\,K = L\,U = \frac{F - X_1}{w\,t\,E}\,p$. Having represented on Fig. 3 the relative displacements of points on the same plate, it is required to find how much Point A_1 will move with respect to Point A_2, B_1 with respect to B_2, and C_1 with respect to C_2.

Imagine that one point of the joint, C_1, is fixed in space. This is quite legitimate since it is the relative displacements of the points of the joint that matter. Then, Points A_1 and B_1 move, as has been seen, to the left from their original positions the distances, $\frac{F}{w t E}\,p$ and $\frac{X_1}{w t E}\,p$, respectively. Points A_2, B_2, and C_2 also move in some manner, a distance as yet undetermined.

If the point, A_2, were fixed, the displacements of B_2 and C_2 would be, respectively, $\frac{X_1}{w t E}\,p$ and $\frac{F}{w t E}\,p$ to the right. Then, the relative displacement of A_1 with respect to A_2 would be $\frac{F}{w t E}\,p$ to the left; the movement of B_1 with respect to B_2 would be $\frac{2\,X_1}{w t E}\,p$ to the left; and the movement of C_1 relative to C_2 would be $\frac{F}{w t E}\,p$ to the left. Point A_2 is not fixed, however; it moves in space in the direction of the force (to the left) an unknown distance, l. Therefore, the displacements are actually: A_1 relative to A_2, to the left, $\left(\frac{F\,p}{w t E} - l\right)$; B_1 relative to B_2, to the left, $\left(\frac{2\,X_1\,p}{w\,t\,E} - l\right)$; and C_1 relative to C_2, to the left, $\left(\frac{F\,p}{w t E} - l\right)$. (Perhaps it would be better to state, "C_2 relative to C_1, to the right" since the assumption has been made that Point C_1 was fixed; however, the two statements are mathematically equivalent.)

The relative displacements, $(A_1\text{-}A_2)$, $(B_1\text{-}B_2)$, and $(C_1\text{-}C_2)$, are measures of the deformations of the rivets, and must be of such magnitudes, as to bring into play the forces distributed among them; namely, X_1 for Rivets A and C, and $(F - 2X_1)$ for Rivet B. Assumed proportionality between the deformation and the force of the rivet gives the necessary equations with the unknowns, l and X_1, as follows,

$$\left(\frac{F}{wtE}\,p - l\right) = k\,X_1 \quad \text{.....................(7)}$$

and,

$$\left(\frac{2\,X_1}{wtE}\,p - l\right) = k\,(F - 2\,X_1) \quad \text{..................(8)}$$

The coefficient of rivets, k, is assumed to be known.

In Fig. 3, if the distance between the lines, 1-1 and 2-2, is $-l$, the deformation of any rivet is equal to the vertical ordinate between the curves, $M\,N\,O$ and $S\,T\,U$, taken under that rivet. This holds true for the case of a lap-joint or a butt-joint with any number of rivets.

From Equations (7) and (8), the force acting on Rivets A and C is,

$$X_1 = \frac{w\,t\,E\,k + p}{3\,w\,t\,E\,k + 2\,p}\,F = C_1 F \quad \text{..................(9)}$$

and the force acting on Rivet B is,

$$X_2 = \frac{w\,t\,E\,k}{3\,w\,t\,E\,k + 2\,p}\,F = C_2 F \quad \text{.................(10)}$$

in which, the coefficients of F, marked C_1 and C_2, represent the fractions of the total force, F, taken up by each rivet in question. When the rivets are very stiff, k is small and approaches zero as a limit; then, $X_1 = \frac{F}{2}$ and $X_2 = 0$. On the other hand, when the rivets are very weak, k is large and approaches infinity as a limit. Then, $X_1 = \frac{F}{3}$ and $X_2 = \frac{F}{3}$; that is, when the rivets are stiff and the plates weak a relatively smaller part of the total force, F, is transmitted by the middle rivet.

If similar lap-joints with four, five, and six rivets, are analyzed, the following expressions are derived for the force acting on rivets:

Lap-Joint with Four Rivets:

Outer rivet,

$$X_1 = \frac{wtEk + 2\,p}{4\,wtEk + 4\,p}\,F = C'_1 F \quad \text{..................(11)}$$

Inner rivet,

$$X_2 = \frac{wtEk}{4\,wtEk + 4\,p}\,F = C'_2 F \quad \text{..................(12)}$$

Lap-Joint with Five Rivets:

Outer rivet,

$$X_1 = \frac{2\,p^2 + 4\,pwtEk + (wtEk)^2}{4\,p^2 + 10\,pwtEk + 5\,(wtEk)^2}\,F = C''_1 F \;\ldots\ldots\ldots (13)$$

Second rivet,

$$X_2 = \frac{pwtEk + (wtEk)^2}{4\,p^2 + 10\,pwtEk + 5\,(wtEk)^2}\,F = C''_2 F \;\ldots\ldots\ldots (14)$$

Middle rivet,

$$X_3 = \frac{(wtEk)^2}{4\,p^2 + 10\,pwtEk + 5\,(wtEk)^2}\,F = C''_3 F \ldots\ldots\ldots (15)$$

Lap-Joint with Six Rivets:

Outer rivet,

$$X_1 = \frac{4\,p^2 + 6\,pwtEk + (wtEk)^2}{8\,p^2 + 16\,pwtEk + 6\,(wtEk)^2}\,F = C'''_1 F \ldots\ldots\ldots (16)$$

Second rivet,

$$X_2 = \frac{2\,pwtEk + (wtEk)^2}{8\,p^2 + 16\,pwtEk + 6\,(wtEk)^2}\,F = C'''_2 F \;\ldots\ldots\ldots (17)$$

Third rivet,

$$X_3 = \frac{(wtEk)^2}{8\,p^2 + 16\,pwtEk + 6\,(wtEk)^2}\,F = C'''_3 F \;\ldots\ldots\ldots (18)$$

A discussion similar to that given for the case of three rivets leads to the conclusion—in the cases of four, five, and six rivets—that for very stiff rivets and weak plates the inner rivets take only a small portion of the total force, but when the plates are strong and the rivets weak, the distribution of forces between rivets in a joint approaches equality.

Example 2.—Butt-Joint with Two Splice Plates and Four Rivets Symmetrical About the Butt-Joint.—Assume that the splice plates in Fig. 4 are of equal thickness, t_2, and different from the thickness, t_1, of the main plate. Let Rivet A transmit to each splice plate the force X_1; and let Rivet B transmit to each splice plate the force, $X_2 = \frac{F}{2} - X_1$. The force exerted by each rivet may be expressed as follows,

$$\frac{F - 2\,X_1}{w\,t_1\,E}\,p - l = k\,X_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

and,

$$\frac{X_1 p}{wt_2 E} - l = k\left(\frac{F}{2} - X_1\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

from which,

$$X_1 = \frac{2\,t_2 p + wt_1 t_2 Ek}{(4\,t_2 + 2\,t_1)\,p + 4\,wt_1 t_2 Ek}\,F = C_1'''' F \ldots\ldots\ldots\ldots (21)$$

and,

$$X_2 = \frac{t_1 p + w t_1 t_2 E k}{(4\,t_2 + 2\,t_1)\,p + 4\,w t_1 t_2 E k}\,F = C_2'''' F \ldots\ldots\ldots\ldots(22)$$

From these equations it follows that if $t_1 = 2\,t_2$, the force is divided equally among the rivets. If $t_1 \neq 2\,t_2$, the rivet on the side of the greater thickness works less.

Example 3.—Case of Butt-Joint with Two Cover-Plates and Six Rivets, Symmetrical About the Butt-Joint.—Following the same logic as in Example 2, the forces transmitted by rivets to each cover-plate are:

$$X_1 = \tfrac{1}{2}\,\frac{2\,p^2 t_2\,(t_1 + 2\,t_2) + 6\,p w t_1 t^2_2 E k + (w t_1 t_2 E k)^2}{p^2\,(2\,t_2 + t_1)^2 + 4\,p w t_1 t_2 E k\,(2\,t_2 + t_1) + 3\,(w t_1 t_2 E k)^2}\,F = C_1''''' F\,..(23)$$

$$X_2 = \tfrac{1}{2}\,\frac{p w t_1 t_2 E k\,(t_1 + 2\,t_2) + (w t_1 t_2 E k)^2}{p^2\,(2\,t_2 + t_1)^2 + 4\,p w t_1 t_2 E k\,(2\,t_2 + t_1) + 3\,(w t_1 t_2 E k)^2}\,F = C_2''''' F\,..(24)$$

and,

$$X_3 = \tfrac{1}{2}\,\frac{p^2 t_1\,(2\,t_2 + t_1) + 3\,p w t^2_1 t_2 E k + (w t_1 t_2 E k)^2}{p^2\,(2\,t_2 + t_1)^2 + 4\,p w t_1 t_2 E k\,(2\,t_2 + t_1) + 3\,(w t_1 t_2 E k)^2}\,F = C_3''''' F\,..(25)$$

in which, X_1 X_2, and X_3 are the forces transmitted to each cover-plate by the rivets, numbered from the outside of the joint.

Comments

Formulas similar to Equations (7) to (25) could be developed for any number of rivets in a butt-joint or a lap-joint with equal or unequal thicknesses of plates and different pitch of the rivets. The principle is always the same, the equations are of the first degree, and their number equals the number of unknowns; but the results are complicated.

One point of distinction between compression joints and tension joints may be noted. In the formulas, w signifies the gross width of the strip only in the case of a compression joint, whereas, in the case of a tension joint, it must be taken as some value between the gross and net widths, depending on the pitch of the rivets, p.

For a small pitch, such as three diameters of the rivet, it is suggested that w be taken as, $w = (W - d)$, or $w = \left(W - \frac{3}{4}d\right)$, in which, W is the gross width. For a large value of p, such as $6d$, for example, it would be reasonable, perhaps, to take $w = W - \frac{d}{4}$, approximately. Possibly the formulas become less accurate for tension when the values of p and w are comparatively small.

The writer has assumed tacitly that cross-sections of the joint remain plane and at right angles to the acting force. This assumption is essentially correct except at the part of the plate immediately surrounding the rivet shank. This factor, however, does not seem to be of such importance as to invalidate the formulas.

PART III.—NUMERICAL VALUES OF THE FORCES OF THE RIVETS

To state that inner and outer rivets do work unequally does not mean much unless the amount of that inequality is known; and in order to find it the numerical value of the rivet coefficient, k, must be determined by experiment. Lacking experimental data, however, an approximate mathematical derivation has been made.

In lap-joints, the coefficient, k, as is evident from Equation (7), is the deformation of a rivet that develops a unit force; in double-strap butt-joints, k is the deformation of a rivet transmitting a unit force to each splice plate.

When deformation occurs at a rivet it is due to one or all of the following three causes: (*a*) The axis of the rivet becomes curved due to shear and moment; (*b*) the surface of the rivet becomes crushed; and (*c*) the bearing surface of the rivet hole becomes crushed and distorted. These three factors combined allow the plates to slip relative to each other. As a result of a rather long mathematical analysis (which is omitted), the following expressions were obtained for the rivet coefficients in a tension lap joint composed of plates of equal thickness, t:

$$k = \frac{1}{7\,315\,d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

and,

$$k = \frac{1}{10\,850\,d \tanh\left(2.3\,\frac{t}{d}\right)} \quad \ldots\ldots\ldots\ldots\ldots (27)$$

Equations (26) and (27) were developed by considering the deformed rivet as a beam with fixed ends, loaded along its length with a continuous load, of a magnitude at each point, proportional to the amount of crushing. For comparatively long rivets, the shear deformation is of little importance, while for the short rivets, on the contrary, the bending deformation need not be taken into account.

Accordingly, Equation (27) takes no cognizance of bending, and, with $\frac{7}{8}$-in. rivets, it should be used for values of $t = \frac{1}{2}$ in., or less, while Equation (26), developed by neglecting shear, should be used with the same diameter of rivets for values greater than $t = \frac{1}{2}$ in.

For a two cover-plate butt-joint with a main plate thickness of t_1 and a cover-plate thickness of t_2, the rivet coefficient, k, must be taken as the average of the two values obtained from Equations (26) or (27), using for t the values, $t = \frac{t_1}{2}$ and $t = t_2$, respectively.

It is not claimed that these formulas for k are very accurate; nor is it believed that any formula could be derived taking all contributing factors into account. It would need to be tested experimentally. The rivet hole may be irregular; the shank of the rivet may not completely fill it; and the driven rivet may leave space around it in cooling. All these factors tend to complicate the problem of determining a mathematically accurate value of the coefficient.

Even allowing for these influences, however, and not taking the friction force of the rivet into account, the coefficient, k, must vary to some extent with the force, since inelastic deformations begin very early because of the high bearing stress on the plate in the middle of the rivet. However, the writer thinks that the values of k in Equations (26) and (27) are not entirely misleading and that they are useful for demonstrating the great differences in the work done by rivets, which ordinarily are assumed to work equally. Table 1 has been prepared for this purpose by means of Equations (9) to (25), using values of k from Equations (26) and (27). In Table 1, C is the fraction of the total force taken by each rivet, according to conventional design (reciprocal of the total number of shears in the joint) and C_1, C_2, and C_3 are fractions of the total force taken by each rivet, as given by Equations (9) to (25), inclusive. The percentage of over-stress is listed in the last column.

TABLE 1.—VALUES OF DISTRIBUTION COEFFICIENTS, C
(Diameter of rivets = $\frac{7}{8}$ in.)

Number of rivets	Width of strip, w, in inches	Pitch of rivets, p, in inches	THICKNESS OF PLATE, t, IN INCHES		Stress factor, k, in inches per kip	DISTRIBUTION COEFFICIENTS				Percentage of over-stress
			t_1	t_2		C	C_1	C_2	C_3	
					LAP-JOINTS					
3....	3	2.5	0.5		1/8200	0.333	0.371	0.257	......	11.4
	3	5.0	0.5		1/8200	0.333	0.395	0.210	......	18.6
	3	2.5	1.0		1/6400	0.333	0.351	0.298	......	5.4
	3	5.0	1.0		1/6400	0.333	0.365	0.270	......	9.6
4....	3	2.5	0.5		1/8200	0.250	0.327	0.173	......	30.8
	3	5.0	0.5		1/8200	0.250	0.368	0.132	......	47.1
	3	2.5	1.0		1/6400	0.250	0.288	0.212	......	15.2
	3	5.0	1.0		1/6400	0.250	0.316	0.184	......	26.4
5....	3	2.5	0.5		1/8200	0.200	0.312	0.140	0.096	56.0
	3	5.0	0.5		1/8200	0.200	0.3618	0.1095	0.0574	80.9
	3	2.5	1.0		1/6400	0.200	0.2570	0.1705	0.1450	28.5
	3	5.0	1.0		1/6400	0.200	0.2955	0.1495	0.110	47.7
6....	3	2.5	0.5		1/8200	0.1667	0.305	0.128	0.067	83.0
	3	5.0	0.5		1/8200	0.1667	0.3596	0.1036	0.0368	115.6
	3	2.5	1.0		1/6400	0.1667	0.241	0.149	0.110	44.5
	3	5.0	1.0		1/6400	0.1667	0.286	0.135	0.079	71.5
					BUTT-JOINTS					
2....	3	2.5	0.5	0.375	1/6180	0.250	0.268	0.232	......	7.2
	3	5.0	0.5	0.375	1/6180	0.250	0.277	0.223	......	10.8
	*	*	0.75	0.375	1/7150	0.250	0.250	0.250	......	0.0
	3	2.5	0.75	0.5	1/7645	0.250	0.262	0.238	......	4.8
	3	5.0	0.75	0.5	1/7645	0.250	0.268	0.232	......	7.2
	*	*	1.00	0.5	1/8200	0.250	0.250	0.250	......	0.0
3....	3	2.5	0.75	0.5	1/7645	0.1667	0.2052	0.1252	0.1696	23.1
	3	5.0	0.75	0.5	1/7645	0.1667	0.2237	0.1003	0.1760	34.2
	3	2.5	1.00	0.5	1/8200	0.1667	0.1860	0.1280	0.1860	11.6
	3	5.0	1.0	0.5	1/8200	0.1667	0.1981	0.1039	0.1981	18.8
	3	2.5	1.0	0.75	1/7200	0.1667	0.2019	0.1363	0.1618	21.1
	3	5.0	1.0	0.75	1/7200	0.1667	0.2211	0.1152	0.1637	32.6

* Any value.

The equations herein presented, with the values given in Table 1, lead to four conclusions, as follows:

1.—The standard practice of dividing the force in proportion to the shearing areas of the rivets and disregarding the deformation of the plates leads to results that are very inaccurate. In a 6-rivet lap-joint the over-stress of an outer rivet designed by the conventional method may be more than 100%, and in a joint with a greater number of rivets, the over-stress seems to be still greater.

2.—Actually, the total force acting on a riveted joint is not distributed equally among the rivets; a larger proportion of the total work is done by the outer rivets.

3.—The proportion of total work in the outer rivets increases as the pitch and diameter of the rivets increase and the cross-section of the plates decreases.

4.—If there are many rivets in a longitudinal row the inner rivets are inefficient, and an increase in the number does not improve the value of the joint appreciably. For example, in Table 1, comparing the values for 5-rivet and 6-rivet joints, it is seen that when the number of rivets is increased from five to six (that is, by 20%), the force on the outer rivet decreases only by 2.3%, 0.6%, 6.6%, and 3.3%, in the four cases given.

It seems likely that in joints containing twenty or more rows of rivets, some of those in the middle perform no work and those on the outside are over-stressed greatly. However, no engineering structure has ever failed because of this lack of uniform distribution, due to the ductile nature of structural steel. After the outer rivets have been stressed beyond the elastic limit, they do not break, but deform almost without increase in stress, and this brings the inner rivets into play. If the rivets were made of some brittle material, the inaccuracy of the conventional design method would soon manifest itself in the failure of the rivets.

Another factor that helps the outer rivets is friction between plates, which has been disregarded entirely in this analysis. Actually, as was seen in the beginning of this paper, the rivets do part of their work by friction which involves no deformation and retards the moment when the stress in a rivet exceeds the elastic limit.

The method of distributing a force through the rivets of a joint as presented in this paper, might prove of practical value in the design of boilers, tanks, and similar structures. The necessary coefficients could be computed quite easily for standard values of pitch, width of plate, thickness of plate, diameter of rivet, and for type of joint. Such data could then be plotted into curves for the uses of the designer. As stated previously, the rivet coefficient, k, would be determined by experiment.

Of course, the greatest disadvantage of the method is that it is less economical than the present standard method of equal distribution. Furthermore, the writer does not recommend it for general structural design of buildings, bridges, etc. In this field, serious complications arise because of the variable pitch and spacing of rivets, because of the use of structural shapes other than plates, because of the irregular shape of gusset-plates, and because of several members meeting at the same point.

However, the writer ventures to offer for discussion a few suggestions concerning the standard design of connections of structural members, in the hope of promoting conditions that will insure more uniform distribution of forces among the rivets, namely:

(1) In compression members, the number of gauge lines should be increased. This would reduce the number of rivets on each gauge line, and hence the inequality of forces taken up by the rivets. Naturally, it would be more difficult to apply this recommendation to the tension members, due to the consideration of the net section.

(2) A minimum allowable longitudinal pitch should be established, since the inequality decreases with the decrease of pitch. Although this is being generally done in common structural practice, sometimes the pitch is lengthened due to a desire to eliminate an extra cut on the gusset-plate.

(3) The number of rivets to be placed on a gauge line should be more or less proportional to the cross-sectional areas at those gauge lines. Thus, in Fig. 5, the number of rivets on Gauge Lines 1 and 5, are just sufficient to develop the angles. These rivets are over-stressed because of the greater cross-sectional areas that are related to these gauge lines, including angles and parts of the plate.

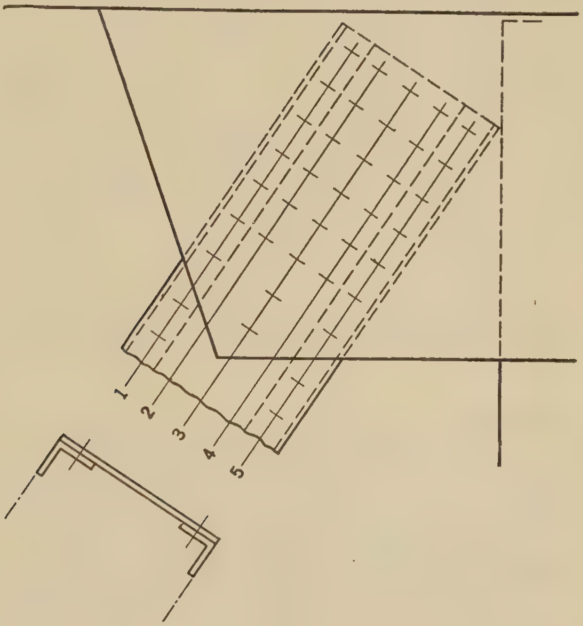

FIG. 5.

ACKNOWLEDGMENT

While this problem has been treated in a general way previously,[2] the methods developed in this paper resulted from independent studies.

The writer wishes to acknowledge his indebtedness to A. H. Finlay, Assoc. M. Am. Soc. C. E., for invaluable guidance and criticism during the preparation of this paper.

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[2] "Theory and Practice of Modern Framed Structures" by Messrs. Johnson, Bryan, and Turneaure, Vol. 3, Chapter 5; also "Structural Engineering" by the late George F. Swain, Past-Presdent, Am. Soc. C. E., p. 407.

DISCUSSION

Henry W. Troelsch,[3] M. Am. Soc. C. E. (by letter).—The problem of the distribution of stress in riveted joints, which Mr. Hrennikoff considers in this paper, is similar in many ways to the problem of the distribution of shear in welded connections, dealt with by the writer in the companion paper.[4] The two are in fact special cases of a more general problem. The only difference is that, for the riveted joints, the connection between the spliced members is assumed to exist only at uniformly spaced points (the rivet centers), while for the welded joints, the connection is assumed to extend uniformly from one end of the joint to the other. In both cases it is assumed that the deformations of the bars and of the connecting members (rivets or welds) are proportional to their stresses.

The similarity of the two theories can be shown by applying the weld theory to several of the riveted joints analyzed by Mr. Hrennikoff, and comparing the results. Let each rivet be replaced by a single weld, equal in length to the pitch, and having the same rigidity as the rivet. The length, z, of the equivalent welded joint will then be the number of rivets multiplied by the pitch. The "detrusion ratio", D, becomes $\frac{1}{kp}$. The shears per unit of length at the rivet centers can then be obtained by Equation (15). These shears multiplied by the pitch will give approximate values of the forces acting on the rivets.

TABLE 2—Comparison of Distribution Coefficients, C, as Determined by Weld Theory and Rivet Theory

Number of rivets	Width of strip, w, in inches	Pitch of rivets p, in inches	Thickness of plate, t, in inches	Rivet coefficient, k, in inches per kip	Detrusion ratio, D, in kips per square inch	Length of equivalent weld, z, in inches	Distribution Coefficients			
							Theory	C_1	C_2	C_3
6......	3	2.5	1	$\frac{1}{6\ 400}$	2 560	15	Weld...	0.239	0.146	0.107
							Rivet..	0.241	0.149	0.110
6......	3	5	0.5	$\frac{1}{8\ 200}$	1 640	30	Weld...	0.344	0.0907	0.0291
							Rivet..	0.3596	0.1036	0.0368
3......	3	2.5	1	$\frac{1}{6\ 400}$	2 560	7.5	Weld...	0.347	0.293	......
							Rivet..	0.351	0.298	......
3......	3	5	0.5	$\frac{1}{8\ 200}$	1 640	15	Weld...	0.373	0.181	......
							Rivet..	0.395	0.210	......

The results for four cases are shown in Table 2, in which the values marked "Weld Theory" were obtained as indicated herein, while those marked "Rivet Theory" are the corresponding coefficients taken from Mr. Hrennikoff's paper. The close agreement between the two theories is brought out even more clearly in Fig. 6, which shows the same four cases. The ordinates to the dotted curves are the shears per inch in the equivalent welds, while the ordinates to the plotted points are Mr. Hrennikoff's rivet coefficients divided by the pitch. Fig. 6 shows that if the assumptions underlying the theories are fulfilled, the distribution of the shear follows the same general law, whether the joint is riveted or welded.

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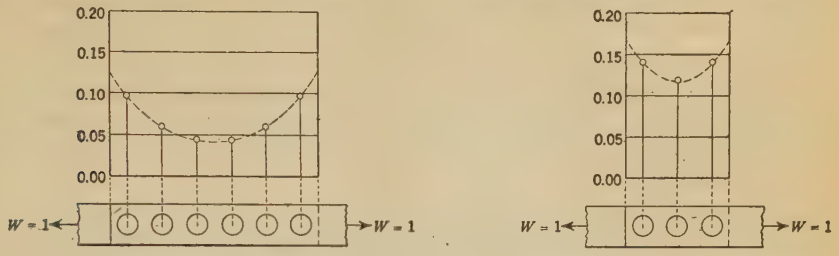
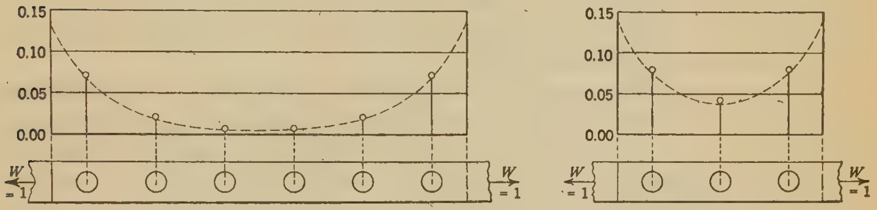
[3] Designing Engr., Am. Bridge Co., New York, N. Y.

[4] "Distribution of Shear in Welded Connections," see p. 409.



(a) 1-in. by 3-in. Bar; $p=2.5$ in., $z=15$ in.

(b) 1-in. by 3-in. Bar; $p=2.5$ in., $z=7.5$ in.



(c) $\frac{1}{2}$-in. by 3-in. Bar; $p=5$ in., $z=30$ in.

(d) $\frac{1}{2}$-in. by 3-in. Bar; $p=5$ in., $z=5$ in.

FIG. 6.—AGREEMENT BETWEEN WELD THEORY AND RIVET THEORY.

HENRY B. SEAMAN,[5] M. AM. SOC. C. E. (by letter).—The analysis presented by Mr. Hrennikoff treats of a subject that has long claimed the attention of designers of riveted trusses, and explains the preference of many engineers for the pin-connected type, even for the shorter spans. In the earlier days of American bridge construction the pin-connected type was used for spans as short as 80 ft., and gave great satisfaction in maintenance; but the preference of the shops for less labor, with heavier tonnage and correspondingly lower shop costs, brought the riveted truss into its present favor.

It was in 1886 that the matter first claimed the attention of the writer, in a riveted lattice girder of about 50-ft. span, for an elevated railway. There were five rivets in the joint, and the writer preferred to place the odd rivet in the center of the cluster, leaving two end rivets to take the initial strain. When the matter was referred to the late Theodore Cooper, M. Am. Soc. C. E., however, he preferred that the single rivet be placed on the outside so as to save material by the cutting of one hole, instead of two holes. This well illustrates the difference between first cost and subsequent cost of maintenance.

The objectionable differences in the stresses of the rivets of a joint, explained by Mr. Hrennikoff, do actually exist in all riveted joints.

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[5] Cons. Engr., Brooklyn, N. Y.

A. H. FINLAY,[6] ASSOC. M. AM. SOC. C. E. (by letter).—In this paper, Mr. Hrennikoff has done a service in calling attention to the weaknesses of the assumptions underlying the common analysis of riveted joints. A periodical review of the assumptions that underlie various structural theories is of the greatest importance, since it is a human failing that, given sufficient time for their assimilation, assumptions at first viewed even with some misgiving eventually tend to be confused with the eternal verities.

In Part I the author has given a clear discussion of the probable behavior of rivets in a joint. Part II translates into the more imposing symbolism of mathematics the assumptions of Part I for the specific case of no friction between the plates.

The wisdom of omitting from consideration the friction between the plates will scarcely be challenged. To any one who has watched rivets being driven with a so-called "bull-riveter" and who has seen the air pressure removed, as is commonly done, within 2 or 3 sec of its first application, the stretching of the still incandescent rivet shank by the spring of the plates upon removal of the pressure, is full of significance.

The writer feels that special credit is due the author for his mathematical evaluation of the proportionality factor expressing the ratio of rivet slip to rivet stress, or k, without which his results would have been merely interesting qualitative relationships. The lack of specific values of this quantity has greatly handicapped previous work in this field. As to the correctness of the author's two values for k, one can only expect that they approach the truth as closely as do the very necessary assumptions, under cover of which the obviously difficult mathematical problem was attacked. The author wisely omits the analysis and modestly refers to it as "rather long."

The writer agrees that the determination of k is a proper field for experiment, and he would be interested to hear of any tests in this connection, although he feels that, in practice, the values of k are as divergent as the conditions of driving.

The general satisfaction given by riveted joints as at present designed as well as the simplicity and economy resulting from them are sufficient guaranties that no radical changes in design will be made. It would be unwise to think of penalizing structural steel for one of its greatest virtues—ductility. How many discordant elements this quality reconciles may never be known, but structural engineers have good reason to appreciate its existence.

With regard to the author's several suggestions for the improvement of riveted joint design, the writer feels that, while in all joints a more uniform distribution of stress will result among the rivets, there is danger of a poorer distribution of stress occurring across the gusset-plate, owing to the more compact rivet groups and the general tendency to bunch rivets which is implied.

F. P. SHEARWOOD,[7] M. AM. SOC. C. E. (by letter).—This paper deals with one of the many problems encountered in structural design whenever riveted joints are closely investigated. They often worry the designers, but by general consent the elasticity and ductility of the joint are depended upon to distribute the strain safely. While such easy method has been recognized and used, it would be more satisfactory to understand and picture what the strains are likely to be, so that designers can proportion and group the rivets and material to obtain the greatest safety.

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[6] Asst. Prof., Civ. Eng., Univ. of British Columbia, Vancouver, B. C., Canada.

[7] Chf. Engr., Dominion Bridge Co., Ltd., Montreal, Que., Canada.

Mr. Hrennikoff has given a clear and useful theory for calculating the rivet strains. His theory is based on perfect elasticity throughout the entire joint. If the initial resistance is from friction, however, and if there is some clearance in the rivet hole, the distribution will not be truly elastic and, when relieved of its load, the joint will not completely recover.

Although the author has referred to friction between the plates, he has disregarded it in his analysis. This may be of the greatest importance, however, in preventing over-stressing rivets in long joints. If there is any clearance between the rivet and its hole, and frictional resistance is about equal to the working value for single shear, the stress at any rivet can not exceed the working value, until the clearance is absorbed by slipping.

The author states that long joints are constantly being used and that none of them has failed. This fact has been proved so often that it has become standard practice to assume that the stress in a joint is equally distributed to all the rivets; that is, that the strength of a riveted joint is the sum of the individual rivet values. It is the general opinion, however, borne out by experience, that connections resisting severe reversals of stress will become loose, and, therefore, must be proportioned differently. These facts indicate that there is some straining effect that is not elastic and that produces wear; and it clearly points to a slip taking place between the plates, or between the plates and the rivet heads. Frictional distortion or slip of the plates caused by stress in the member will not recover on the removal of the stress, unless there is as great a difference between the strains in the plates as that which caused the slip.

To allow slip to take place without shear in the rivets there must be clearance between the rivet and the hole. That this exists seems very probable. When a rivet is driven its heat temporarily reduces the size of the hole. The rivet is upset to the size of the heated hole, and pressure is released from both the hole and the rivet when they are at a high heat. Both the hole and the rivet must cool, and, therefore, a clearance is likely. As an illustration, let it be assumed that when the driving of a rivet is completed there is an average temperature of 500° above normal temperature in the edges of the hole at the shank of the rivet; then the clearance for a $\frac{7}{8}$-in. rivet will be approximately: $0.0000065 \times 500 \times 2 \times 0.875 = 0.00525$ in. That means that the plates can move past each other 0.005 in. before the rivet touches the walls of the hole and it is strained in shear or bearing.

The amount of frictional resistance will depend on the initial tension developed in the rivet by cooling. It seems probable that this tension on the rivet head is about equal to the elastic limit of the material, say, 35 000 lb per sq in. By assuming the coefficient of friction at 20%, the resistance at each surface will be $35\,000 \times 0.6 \times 0.2 = 4\,200$ lb.

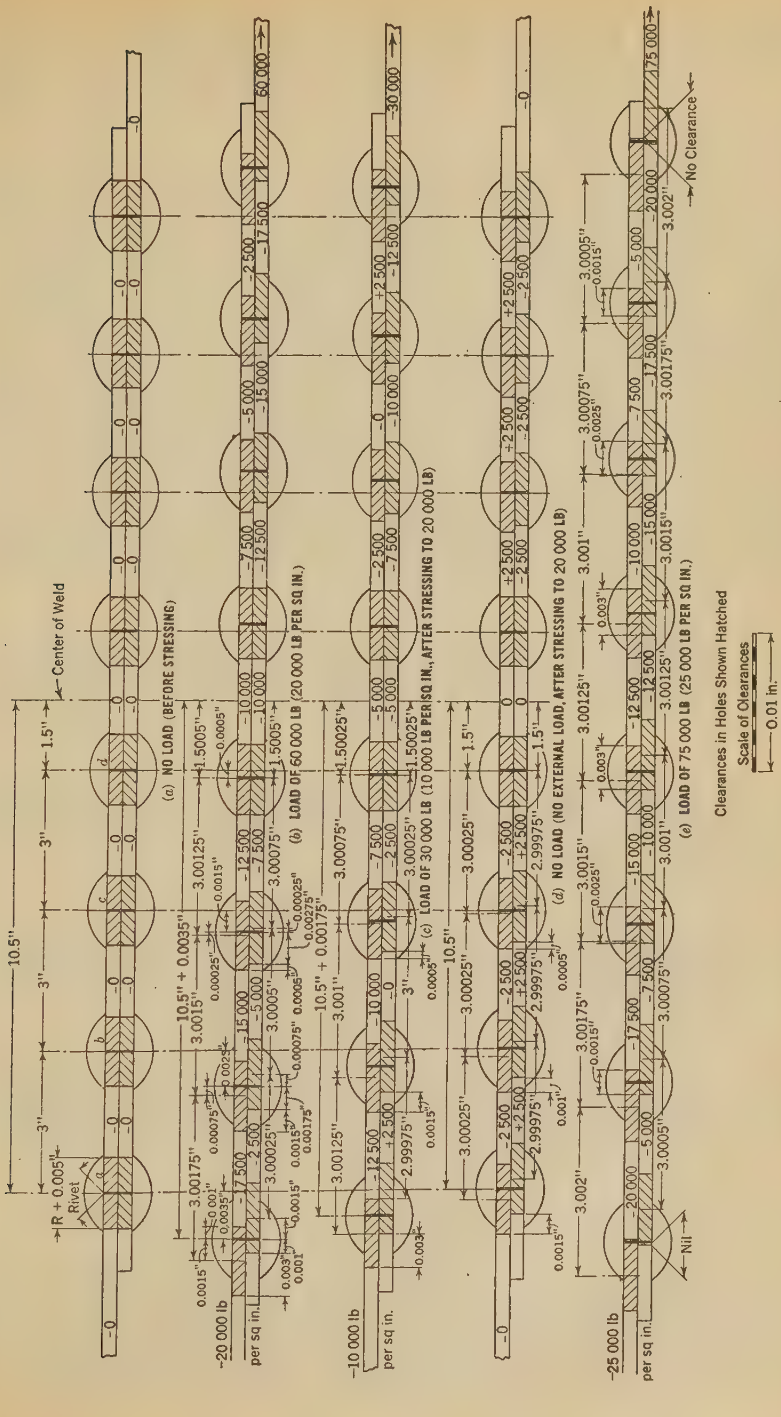
To enable the plates to move, at least two surfaces are affected; therefore, the force required to overcome friction will be equal to about 8 400 lb per rivet. Fig. 7 illustrates the clearances, relative positions, and strains in the holes of an 8-rivet lap-joint, under various conditions of loading. The joint is composed of two 1-in. by 3-in. plates. In order to simplify the computations, the force required to overcome friction is assumed equal to 7 500 lb per rivet, or 2 500 lb per in. of width, and $E = 30\,000\,000$ lb per sq in. The plates are shown extended an amount equal to 0.00033 in. per 10 000-lb strain, or 0.001 in. per 10 000-lb strain between rivets. Stresses are given in pounds per square inch of material. Fig. 7(*a*) illustrates the clearances in the holes to a very exaggerated scale. The total clearance between the hole and rivet is assumed to be 0.0025 on each side, or 0.005 in all.

Fig. 7(*b*) gives the relative positions and strains of the two plates when a stress of 20 000 lb per sq in. of the main plate is applied (full section is assumed at holes). When this load is applied, the frictional resistance of one rivet will be exerted at Rivet *a*, but as this is insufficient to equalize the strain in the two plates they will slip past each other, and a load of 17 500 lb per sq in. will remain in the upper plate until the resistance of Rivet *b* is met, where another 2 500 lb per lin in. (the resistance from one rivet) will be transferred to the lower plate, and so on, until the strains in the two plates are equal.

The difference of deformation in the upper and lower plates between the point where the strains are equal and any rivet in the connection will give the amount of slip at that point. In this example, the slip at Rivet *a* equals 0.003 in., leaving a clearance of 0.001 in. between the hole and rivet. At Rivet *b*, it is 0.0015 in., and at Rivet *c*, 0.0005 in., while at Rivet *d*, there is no slip because the strains in the plates are equal. Some clearance still remains at all these rivets and, therefore, no shearing can have been developed in the rivets themselves. The bending resistance in the rivet due to its heads slipping on the plates cannot be very much. If it is disregarded there can be no stress of any kind in the rivet, until the slip of the end rivet has exceeded the clearance in the hole.

Fig. 7(*c*) illustrates the positions and strains when 50% of the stress is removed. No alteration in the relative positions of the two plates is necessary to keep the friction below the slipping value of 2 500 lb and, therefore, no slip or wear results. When more than 50% of the load is removed the plates must slip to reduce the friction to 2 500 lb. Fig. 7(*d*) shows the positions and strains resulting when all the stress is removed, and it shows also that Rivets *a* and *b* have partly slipped back. The 20 000-lb unit strain assumed in this example does not often occur in actual practice, and it is probable that the ordinary range of strains developed will not be sufficient to slip the plates back and loosen the rivets by wear.

When the stress exceeds the frictional value of all the rivets the entire connection will slip until the end rivets come into contact with the walls of the holes. These end rivets must resist the excess of stress over the friction value of all the rivets until the deformation of these end rivets is sufficient to bring the next rivet into contact.


10.5"
R + 0.005"
Rivet
3"
3"
3"
1.5"
Center of Weld
a
b
c
d
-0
-20 000 lb per sq in.
60 000
-17 500
-15 000
-12 500
-10 000
-7 500
-5 000
-2 500
10.5" + 0.0035"
3.00175"
3.0015"
3.00125"
1.5005"
3.00075"
3.0005"
3.00025"
-10 000 lb per sq in.
30 000
-12 500
-10 000
+2 500
10.5" + 0.001 75"
3.00125"
3.001"
3.00075"
1.50025"
2.99975"
3.00025"
-0
2.99975"
+2 500
-2 500
3.00025"
1.5"
10.5"
-25 000 lb per sq in.
175 000
-20 000
-17 500
-15 000
-12 500
-10 000
-7 500
-5 000
3.002"
3.00175"
3.0015"
3.00125"
3.001"
3.00075"
3.0005"
Nil
No Clearance
0.0015"
0.003"
0.001"
0.0035"
0.00075"
0.00025"
0.0005"
0.0025"
0.00275"

(*a*) NO LOAD (BEFORE STRESSING)

(*b*) LOAD OF 60 000 LB (20 000 LB PER SQ IN.)

(*c*) LOAD OF 30 000 LB (10 000 LB PER SQ IN., AFTER STRESSING TO 20 000 LB)

(*d*) NO LOAD (NO EXTERNAL LOAD, AFTER STRESSING TO 20 000 LB)

(*e*) LOAD OF 75 000 LB (25 000 LB PER SQ IN.)

Clearances in Holes Shown Hatched

Scale of Clearances

0.01 in.

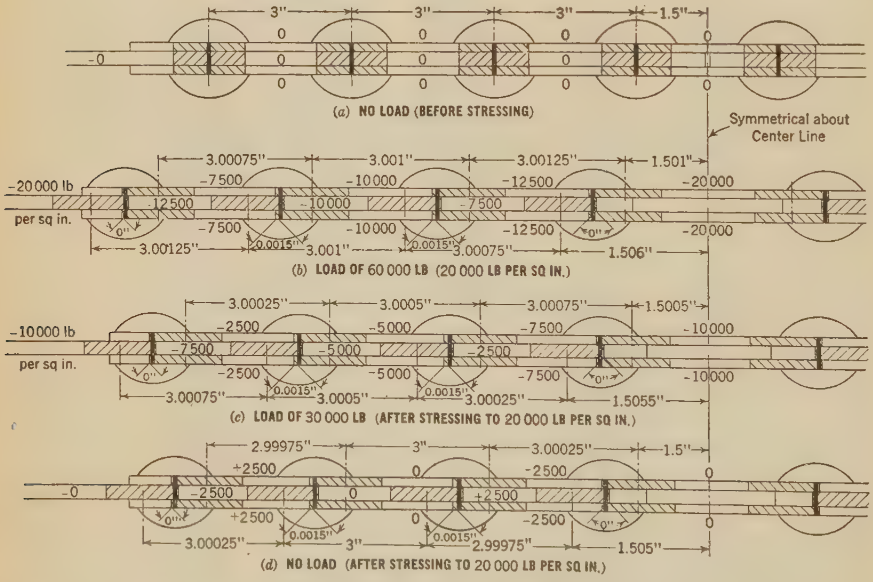
FIG. 7.—CLEARANCES, RELATIVE POSITIONS, AND STRAINS IN HOLES OF AN 8-RIVET LAP-JOINT.

Fig. 7(*e*) gives the clearances in the holes when this connection is stressed to slightly beyond the total friction value of the eight rivets. The rivets at each end must transfer 5 000 lb per sq in., which is twice the assumed friction value of the rivet friction; but there is still clearance at the second rivets. (No allowance has been made for the deflection from shearing and bearing of the end rivets.)

For the sake of simplifying the general picture of slippage, these examples of lap-joints are based on the assumption that the rivet heads slip on the plates at one-half the resistance and displacement that the plates slip past each other. The resistance from the bending of the rivet is also neglected.

If a butt-joint is examined it will be found that the same general slipping takes place, except that the plates will first slip on one another until the rivet shank bears against the inner plate. The heads will then slip in the outer plates, and two extra slipping surfaces will be brought into resistance, until the end rivet is jammed between the three plates and any further resistance must be entirely taken up by the end rivets in shear until the deformation is sufficient to bring the next rivet to bear on the walls of its hole.

Fig. 8 is a corresponding example of a butt-joint. The force required to overcome friction is assumed to be equal to 3 750 lb per rivet on each surface, or 1 250 lb per rivet on each surface per inch of width. As in Fig. 7, stresses are given in pounds per square inch of material. It can be readily seen that the friction between the plates at all the rivets and the friction between the rivet heads and plates at Rivets *a* and *d* are just sufficient to resist the load, while at Rivets *b* and *c* some clearance still remains. Any addition to the


3"
3"
3"
1.5"
-0
0
0
0
0
Symmetrical about Center Line
3.00075"
3.001"
3.00125"
1.501"
-20 000 lb per sq in.
-7500
-10 000
-12 500
-20 000
-12500
-10000
-7500
0"
0.0015"
3.00125"
3.001"
3.00075"
1.506"
3.00025"
3.0005"
3.00075"
1.5005"
-10 000 lb per sq in.
-2500
-5000
-7500
-10 000
-7500
-5000
-2500
0.0015"
3.00075"
3.0005"
3.00025"
1.5055"
2.99975"
3"
3.00025"
1.5"
+2500
-2500
+2500
0
0.0015"
3.00025"
2.99975"
1.505"

(*a*) NO LOAD (BEFORE STRESSING)

(*b*) LOAD OF 60 000 LB (20 000 LB PER SQ IN.)

(*c*) LOAD OF 30 000 LB (AFTER STRESSING TO 20 000 LB PER SQ IN.)

(*d*) NO LOAD (AFTER STRESSING TO 20 000 LB PER SQ IN.)

FIG. 8.—CLEARANCES, RELATIVE POSITIONS, AND STRAINS IN HOLES OF AN 8-RIVET BUTT-JOINT.

load will be entirely taken by Rivets *a* and *d* in shear until the deformation of those rivets amounts to the clearances remaining at Rivets *b* and *c*. In the case of the butt-joint the resistance to recovery when the load is removed is greater than in the case of the lap-joint.

The stresses and dimensions of clearances are only approximate and are used to illustrate, in the simplest way, the possible workings in a riveted joint, which has minute clearances in the holes when under stress. They also illustrate the fact that very unequal distribution of stress among the rivets of a group is not probable in the usual length of riveted connections.

The problem of how much the frictional resistance amounts to and what range of stress can be used without producing constant slip, depends on a great number of variable uncertainties and would be difficult to solve by mathematics. Experience indicates that the range can be almost up to reversal without producing wear, and general practice has largely assumed this in designing riveted structures. The main problem is, as to whether there is any clearance in the rivet holes, and, if so, how much?

The writer has endeavored to show: First, that there is clearance between the hole and the rivet and that this clearance must be absorbed before the rivets can take any shear; and, second, that slipping of the connected plates occurs and that the force to produce this movement is consistent; that is, it does not vary with the amount of movement. If these are the facts and the deformation required to equal the clearance is sufficient to absorb the total strain in joints of usual length, Mr. Hrennikoff's theory, which is based on the elasticity of the rivets and material, does not function, except for a small part of the stresses in very long joints.

W. P. Roop,[8] M. Am. Soc. C. E. (by letter).—It is an opportune moment for re-opening discussion of riveted joints. Comment of two kinds is presented herewith, namely, another solution of the specific problem of this paper, and a more general discussion of experimental methods and data.

The consideration of relative deformations in different parts of a riveted joint is an important departure from practice in conventional design, and one which leads to some rather unfamiliar points of view.

Point Rivets.—The author's conception of a rivet appears to be that of a point or small area in which there is no relative motion of the faying surfaces with respect to each other at loads below limiting friction; when this limit is exceeded relative motion occurs to the extent that resistance due to friction and to elastic deformations in plates and rivet will permit. This elastic resistance is considered to be proportional to the amount of the relative motion, or slip.

From this as a starting point, the author develops formulas for action of rivets in a lap-joint in tension. Couples due to the eccentricity of the tensions in the two plates are ignored, which is more or less equivalent to splitting a double-strapped joint down the center of the plate, so as to obtain only one strap in the picture. Fig. 3 shows the assumed relative deformations.

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[8] Lieut. (CC), U.S.N.; New York Shipbuilding Co., Camden, N. J.

Diagrams of this kind form a convenient basis for the entire discussion, and their character should be defined a little more clearly. It is preferable to adopt the mid-length of the lap as the point of reference, and in order to obtain complete symmetry this point is taken half-way between the points at mid-length of the two sides of the lap. On Fig. 3 this point would be represented midway between Points Q and W. Distances in the diagram measured vertically from a horizontal line through this point represent horizontal displacements, and as the diagram is drawn, distances laid off upward represent displacements to the left, and *vice versa.*

A somewhat different graphical representation of this situation is obtained by drawing two parallel lines representing the two sides of the surface of contact (faying surface) in the lap. Although actually these two surfaces coincide, the lines representing them are separated, in Fig. 9, by a convenient distance. To each point in one plate, P, there corresponds a point, P', in the other plate; in the unloaded condition of the joint these two points coincide, and in Fig. 3 a line joining them would be normal to the base lines, but when the joint is loaded, all these lines are inclined. Conventional design is based on the idea that the inclination, $P\,P'$, is uniform throughout the joint, so that all rivets are equally loaded. However, this is impossible except in a limiting case, such as that where contact between the plates is at two points only, the load being equally shared at these two points. Otherwise, the relative displacement of one plate with respect to the other is more complex.

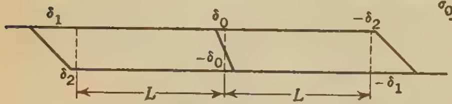

FIG. 9.—DEFLECTIONS

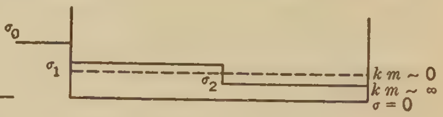

FIG. 10.—STRESSES

In a diagram of the type used by the author the case of two (rows of) point rivets would be represented by two horizontal lines at equal distances above and below the base. The case of three point rivets is that of the author's Example 1.

Graphical interpretation may be given to the results expressed in Equations (9) and (10). When k is small and the rivets are stiff, the lines, $M\,O$ and $S\,U$ (Fig. 3), are nearly straight. The value of l is also affected, and in the limiting case of $k = 0$, becomes equal to $F\frac{p}{wtE}$. The middle row of rivets then carries no load, and $M\,O$ and $S\,U$ are horizontal. As k increases, and the connections in the lap become more flexible, $M\,O$ is inclined and N departs from the straight line joining M and O. When $k = 3\frac{p}{wtE}$, l has already reached a value slightly negative, namely, $-\frac{1}{11}\,F\,\frac{p}{wtE}$, and as k continues to increase, l will reach larger negative values, approaching $-\frac{kF}{3}$, and the inclination of $M\,O$ will increase indefinitely.

It is for such reasons that the writer prefers another type of diagram instead of that used by Mr. Hrennikoff. In this third type of diagram the horizontal co-ordinate is still length, measured both ways from mid-length of the joint. Let the entire length of the joint be $2L$, and the distance from mid-length to a given section of the joint, ξ; then a convenient co-ordinate to use is $X = \frac{\xi}{L}$, since it is applicable to a joint of any length, and the joint lies between the limits, $-1 < X < +1$. For a vertical co-ordinate, however, the writer chooses stress in the plate. In the author's notation, this is $\frac{F}{wt}$, and will be denoted by σ, with subscripts, 1 and 2, for the upper and lower plates. The stress in either plate outside the joint is σ_0, and a moment's reflection will show that in a lap of plates of equal thickness, $\sigma_1 + \sigma_2 = \sigma_0$, at all values of X. With any symmetrical arrangement of the connections, at $X = 0$, $\sigma_1 = \sigma_2 = \frac{\sigma_0}{2}$. Between point rivets, σ_1 and σ_2 will have uniform values, so that the curve will be, for σ_1, a series of steps downward from σ_0 at $X = -1$, with a riser at each intervening rivet, landing on $\sigma_1 = 0$ at the last rivet on the right, where $X = +1$.

With two rows of rivets, σ_1 drops immediately at $X = -1$ to $\frac{\sigma_0}{2}$ and retains this value until $X = +1$, when it drops to 0. With three rows of rivets expressions for σ_1 and σ_2 may be obtained by a process similar to that used by the author. The deflection of a point rivet, δ, is proportional to the tensile stress (in the plate) which produces the deflection, thus $\delta = k\sigma$. This tensile stress also produces elongation in the plate according to an equation of the type, $d = \frac{\sigma}{m}$. Thus, k is the inverse stiffness of the rivet in shear, and m is the elastic modulus of the plate. The solution is,

$$\sigma_1 = \sigma_0 \frac{2\,k\,m + 1}{3\,k\,m + 2}$$

When k and m are large, the rivets slender and flexible, and the plates thick, σ_1 is nearly two-thirds σ_0, and each of the three rivets carries about one-third the total load. When k and m are small, the rivets large, and the plate thin, σ_1 is about one-half σ_0, and the middle rivet carries little load. Fig. 10 exhibits this graphically. The stress in the other plate at the same point, X, is σ_2, or, on account of symmetry, it is the stress in the same plate at $X' = -X$.

By similar methods a solution can be found for a joint with any number of rows of point rivets, but the elimination of deflections will require solution of a correspondingly large number of linear equations. This will lead to a uniform value for σ_1 in each interval of plating between rows. These values will drop from σ_0 to 0 by a corresponding number of steps; when $k\,m \sim \infty$, the load will be shared about equally by all rivets, and when $k\,m \sim 0$, the outer rivets will take practically all of it.

Distributed Connection.—Now the point rivet, on which this entire analysis is based, is a rather crude abstraction. The area of the holes alone in a lap-joint may run to 13% of the area of the lap, and the area under high heads and points may double that figure. Further, the clamping effect of a rivet is surely not limited to the area under the head. In studying this question the writer has approached it with the assumption that at moderate loads the clamping area is 100% of the lap, and that as slip sets in, it starts at the edges where the stress concentrations occur, and spreads inward. He has further assumed that even in the area of contact the deflections in the parting between the plates are large compared with the variations in stress through the thickness of the plate. Such an assumption is necessary, in order to make any analysis of this character possible, but instead of connection between the plates only at isolated points, assume the elastic resistance to tension to be distributed continuously over the entire area of the lap.

The full analysis for this case has been worked out, but only the results and definition of the quantities will be given herein. The stiffness of the elastic connections must now be expressed in terms of elastic resistance per unit length of the joint (parallel to the load), and also, of course, per unit deflection. The quantity, k, will be defined, as before, in an inverse relation, so that a high value of k is associated with large deflections under a given load; m retains the same meaning as before, being in direct proportion to Young's modulus for the plate, but depending also on dimensions of the joint. The solution is in the form,

$$\sigma_1 = \frac{\sigma_0}{2}\left[1 - \frac{C}{C^2 - 1}\left(C^X - C^{-X}\right)\right]$$

As before, when $X = 1$, $\sigma_1 = \sigma_0$; when $X = 0$, $\sigma_1 = \frac{\sigma_0}{2}$; but instead of changing by a series of steps, the stress now varies continuously, following a curve the course of which depends on the value of C. This, in turn, depends on the elastic characteristics of the joint according to the equation:

$$C = e^{\sqrt{\frac{1}{km}}}$$

Thus, when km is large, the elastic connection flexible, and the plates thick, C approaches unity, log C approaches zero, and C^X approaches $1 + X \log C$. Then,

$$\sigma_1 = \frac{\sigma_0}{2}\left[1 - \text{Constant } X\right]$$

and since $\sigma_1 = \sigma_0$ when $X = -1$, and $\sigma_1 = 0$ when $X = +1$, the constant must be -1. On the other hand, when km is very small, C is very large, and:

$$\sigma_1 \sim \frac{\sigma_0}{2}\left[1 - C^{X-1} + C^{-X-1}\right]$$

It is easy to see how σ_1 varies with X at various values of km by writing a few numbers. For example, when $km = 0.01$, σ_1 has dropped off to 0.57 at $X = -0.8$; and as $km \sim 0$, the curve would approach a vertical drop to

$\sigma_1 = \frac{\sigma_0}{2}$, following along this value until the other end of the joint was approached. In Fig. 11, $\frac{\sigma_1}{\sigma_0}$ is plotted on X for various values of km.

This solution appears to make a somewhat closer approach to the truth than one based on point rivets, but it still departs from the truth in assuming that the elastic connections between the plates are distributed all over the area of the lap. The connections are doubtless actually somewhat spotty, and actual stress distributions would be affected. Nevertheless, the main feature of the results must hold in actual cases also, such as a concentration at the edges of the lap which always exists, but is more pronounced when plate edges are clamped tightly and the plate itself is relatively extensible. Over-all extensions under load have also been calculated both for point rivets and distributed clamping. Discussion of this matter, however, is reserved for the present.

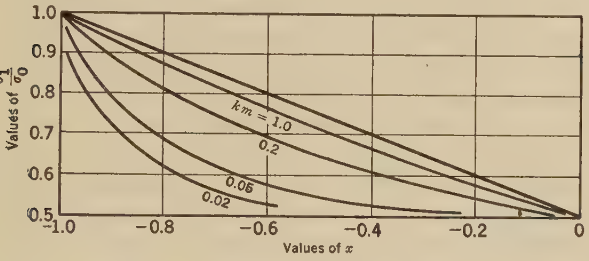

FIG. 11.—DISTRIBUTED CONNECTIONS.

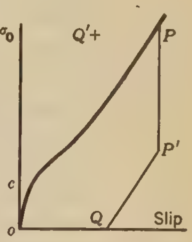

FIG. 12.—SLIP CURVE.

The next step in the analysis of a lap-joint would lie in consideration of the distribution of stress across the thickness of the lapped plates, which obviously cannot be uniform. This would lead still deeper into the theory of elasticity, although the question is also accessible to study by the photoelastic method. Purely analytical study of riveted joints in this field appears to have about reached its present limit. In fact, the object of such calculations as these is not to obtain conclusions directly applicable in the design of joints, but rather to obtain a nominal analysis sufficiently close to the truth to permit its use in reduction of experimental data and evaluation of empirical constants.

Experimental Studies.—All experiments have from necessity dealt with entire joints and not with such abstractions as point rivets. Perhaps it may become possible to study experimentally the separate elements of a riveted joint, but for the present designers are practically limited to judging the correctness of analysis by the extent to which it leads to correct reconstruction of data observed for whole joints. Such data have been available for a long time as far as ultimate strength is concerned, but, unfortunately, other data which are of more significance than the ultimate strength, have been obtained only to a limited degree. By this are meant in particular data on stresses at working loads, and on elongations. That these are more significant than ultimate strength of isolated joints will be apparent from the consideration that (at least in ships) a joint seldom works in isolation, but is almost always in combination with other elements of construction in such a way that relative deformations are of primary significance. Further, after elastic limits have been passed, even an isolated joint works in an entirely different way from that by which working loads are resisted. No doubt the basic assumption of conventional design, that all rivets share the load equally, has some justification as far as the ultimate strength of a lap-joint goes. For such a joint as that shown in the author's Fig. 2, however, such an assumption is not even approximately correct, and even in the simple lap-joint, under working loads, inequalities in rivet loading, such as the author points out, have important consequences.

Experiments in which elongation and stress distribution were observed, were contemplated as long ago as 1869.[9] Known to the writer in detail are only two series of tests in which serious efforts to observe elongations were made, namely, those of Montgomerie,[10] and those made at the Bureau of Standards by Commander E. L. Gayhart, for the Bureau of Construction and Repair, U. S. Navy Department,[11] which will be referred to as the "1924 tests." No doubt similar studies have been made, and are known to others.

In all discussion of the elongation of riveted joints prominence is given to the word, "slip." It is important that its meaning should be agreed upon. In early tests a fine line was scribed across the parting at the plate edges and observed with a glass. When the continuity of this line was observed to be broken, slip was said to have begun. Close observations of slip on this basis, taken on a series of lines spread over the length of the joint, with the help of a micrometer microscope, might produce results of value, and as far as the writer knows this has never been undertaken.

Since over-all elongation of the joint is, for many purposes, the significant quantity, slip has sometimes been taken to be the remaining elongation on release (but not reversal) of the tensile load. This is based on the idea that this phenomenon is due to having exceeded a critical frictional resistance somewhere, which permitted sliding of the plates over each other; release of load would not be sufficient to remove such a deformation which would thus appear as a permanent set. For observation of slip on this basis, however, release is necessary, leading to complicated re-adjustments. For this reason a third view of slip was taken in the 1924 tests.

Instead of simple over-all elongation, deformation was measured as between a number of pairs of points; curves were plotted on load, and a break in the curve, such as that marking proportional limit in a tensile test specimen, was taken to mark the beginning of slip. The amount of slip at a given load between a given pair of points was obtained by subtracting from the extension as observed by a dial gauge a calculated correction for the elastic part of this extension. This calculation was on a rather nominal basis, no great emphasis being placed on its absolute value in view of the numerous other uncertainties involved.

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[9] "Shipbuilding in Iron and Steel," by E. S. Reed; also, "On the Present State of Knowledge as to the Strength and Resistance of Materials," by Jules Gaudard, *Minutes of Proceedings*, Inst. C. E., Vol. XXVIII (1868–69), pp. 536–571, and Vol. XXIX (1869–70), pp. 25–97.

[10] *Transactions*, Inst. of Engrs. and Shipbuilders in Scotland, 1920; also, *Transactions*, Inst. of Naval Archts., 1923.

[11] *Transactions*, Soc. of Naval Archts. and Marine Engrs. (1926), p. 55.

In order to eliminate as far as possible arguments based, not on real differences of opinion, but on differences of definition, the writer proposes that slip be regarded as the kind of a quantity actually observed in the early tests on the plate edges, namely, the distance separating two points on the two sides of the faying surface which were originally coincident. On this basis slip will depend on the entire history of the joint, so that "original condition" means whatever starting point was adopted for the observations. On this basis, slip throughout the area of the joint is not directly amenable to observation. Such data on slip as are inferred from observations should be qualified; thus "edge" slip explains itself. Inferred slip based on the subtraction of a calculated elastic correction might be called "nominal" slip. As defined, slip will vary from point to point throughout the joint; values obtained from data on the extension of the whole joint might be called 'over-all" slip. Perhaps the most important distinction of all is that between residual slip, which is that remaining on release of load, and elastic slip which, generally speaking, would be in direct proportion to the load. More specifically, elastic slip would be defined as the difference between the total slip and the residual slip, relative motion of the plates being under control of the elastic resistance of the rivets.

Armed with this specific view of the nature of slip, the reader may return to view Part III of the author's paper, devoted to "Numerical Values of the Forces of the Rivets."

Accepting the author's demonstration of overstressing in outer rivets, it still seems fair to say that the picture bears little resemblance to the facts until phenomena of friction and slip are considered. Commander Gayhart has shown[12] that over-all slip begins at loads that may well lie within working ranges. The curves as drawn in his paper require attention to the manner in which they are constructed. A type curve is shown in Fig. 12, in which it should be noted that the point plotted at Q' is obtained only at zero external load; it is actually located at Q, but is placed at Q' for convenience. During release the course of the curve is from P to Q. The vertical descent, $P P'$, represents the release of the frictional resistance. However, before the elongation can be reduced, as in passing from P' to Q, a reversed friction must be overcome. The friction, $P P'$, therefore, might be expected to equal twice the critical friction, $O C$. In the case shown in Commander Gayhart's paper it is much greater than this.

Beyond the point at which over-all slip begins, not only elastic slip, but residual slip, increases approximately in direct proportion to total load. These facts lead to questions as to the details of slip action, which must be settled somehow before a theory of riveted joints has even a chance of being correct.

If it could be assumed that friction played no part at loads above that of initial slip, or if the residual slip could be assumed equal to zero, the theory developed by the author would be applicable to the idealized cases considered. Such assumptions, however, are obviously wide of the mark. It is not possible to separate elastic slip from residual slip and then treat the elastic slip as if residual slip played no part in determining elastic reactions. The true analysis of these curves must be based on a separation, not of the elements of slip under a given total load, but of the elements of load under a given total slip.

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[12] See *Transactions*, Soc. of Naval Archts. and Marine Engrs. (1926), Pl. 22, Sheet 3.

Summary and Conclusion.—The slip curve which has been discussed is not the only type that was obtained in the 1924 tests, but this is not the place for extended discussion of those tests. The principal object of this comment has been to show that rivet action, even to the first degree of approximation, may be considerably more complicated than is contemplated by the author. If sound progress beyond what he aptly calls "conventional design" is to be made, it is essential that more experimental information be made available, and especially that it should receive a searching analysis, with subsequent tests especially devised to provide answers to the questions developed by the analysis. When the results of such study become accessible to those who are directly responsible for detail design the Engineering Profession may perhaps hope to see no more such absurdities as that mentioned by the author—a tensile joint with twenty rows of rivets.

H. N. HILL[13] AND MARSHALL HOLT,[14] JUNIORS, AM. SOC. C. E. (by letter).—The unequal distribution of the load on the rivets in a joint is a matter deserving of greater recognition than is ordinarily accorded it by the Engineering Profession. This distribution has been the subject of an investigation conducted by the writers under the supervision of R. L. Templin, M. Am. Soc. C. E. The investigation concerned the distribution of load in butt-joints comprising splices between plates of steel and a high-strength aluminum alloy, using both steel and aluminum alloy rivets.

The mechanical properties of the two metals used in the investigation are given in Table 3. It may be noted that there is little difference between the strength of the two materials, but that the modulus of elasticity for the aluminum is about one-third that for steel. The problem involved the distribution of the load on the various rivets, not only as affected by the physical dimensions of the joint, but also as affected by the different values of the modulus of elasticity for the two materials.

TABLE 3.—MECHANICAL PROPERTIES OF STEEL AND HIGH-STRENGTH ALUMINUM ALLOY

Properties	Steel	Aluminum alloy
Ultimate tensile strength, in pounds per square inch	59 000	56 000
Yield strength in tension, in pounds per square inch	28 000	33 000
Percentage elongation in 2 in.	37.5	23.0
Modulus of elasticity, in pounds per square inch	29 000 000	10 000 000
Shear strength, in pounds per square inch	45 000	35 000
Shearing modulus, in pounds per square inch	12 000 000	3 800 000

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[13] Testing Engr., Aluminum Co. of America, New Kensington, Pa.

[14] Research Engr., Aluminum Research Laboratories, Arnold, Pa.

The problem was attacked in two ways: First, an "elastic theory" was developed, along lines similar to those described by Mr. Hrennikoff, taking into account the elastic deformation of the various parts of the joint and assuming the load transmitted by each rivet to be proportional to the relative displacement of the plates; and, second, a number of double-strap butt-joints, embodying a single longitudinal row of rivets, were fabricated and tested to determine experimentally the manner in which the load was distributed among the various rivets.

In establishing a relationship between the deformation and the load on a rivet, the writers have not felt that the limits of accuracy, dictated by the uncertainties existing in any joint, warrant an elaborate mathematical treatment. An experimental determination of the relationship between deformation and load would be of considerably more value than any elaborate mathematical analysis, particularly since the theoretical treatment would require the substantiation of experimental data to make it convincing.

As a basis for the calculations in the mathematical treatment, a relationship between the load and deformation on a rivet was obtained from a preliminary test of a joint, similar to those to be investigated, but having only one rivet on either side of the splice. This specimen was fabricated entirely of aluminum alloy, and consisted of two main plates, 1 in. thick by 5¾ in. wide, two cover-plates of the same width and ½ in. thick, and two ¾-in. rivets. The relative movement of the plates was measured for different loads on the rivets. Although for low loads and relatively high loads, the load-deformation curve was not a straight line, for loads corresponding to the range to be encountered under normal working conditions, differences in the slopes of lines drawn through zero from points on the curve corresponding to the different loads are not great enough to produce an appreciable effect on the results of the calculations.

The results of this test established a relationship between the load and deformation of an aluminum alloy rivet in plates of the same material, but did not cover the other conditions in the joints to be tested. Since the deformation is partly produced by yielding of the plates, a definite load on an aluminum rivet in aluminum plates will produce a greater relative movement than would occur if the plates were of steel. Since the modulus of elasticity of aluminum, is approximately one-third that of steel, it is reasonable to assume that, in a steel specimen of the same dimensions as the one tested, the ratio of deformations would be 1 to 3.

In Table 4 the ratios of load to deformation for conditions other than those covered by this test have been determined by estimating the effects of differences in the conditions. Familiarity with the calculations for determining the distribution of the load among the various rivets reveals the fact that the answer is not extremely sensitive to variations in the ratio of load to deformation of a rivet. Any reasonably correct ratio of load to deformation will give results that are within the range of accuracy to be expected from variations introduced in the fabrication of a particular joint.

It is interesting to note the agreement between the coefficient determined experimentally by the writers and that calculated from the author's equations for the same case. The larger of the two values calculated according to Equations (26) and (27) for the second set of conditions (Table 4) was 0.000182 in. per kip, from Equation (26). Since the author's coefficient for the double-strap butt-joint represents the ratio of the rivet deformation to the load transmitted to one splice plate, and the values in Table 4 are the ratios of rivet deformations to the total rivet loads, in order to make the two comparable it is necessary to divide the author's value by two. This gives the value of 0.000091 in. per kip, whereas a value of 0.00012 was obtained by the writers as one-third of the relative movement determined experimentally for the first set of conditions of Table 4, one-third being the ratio of the modulus of elasticity of aluminum to that of steel.

TABLE 4.—Relations Between Deformation and Load on Rivets

Thickness of plates, in inches	Main plate	Cover-plate	Rivets	Coefficient $\left(\frac{\text{Inches}}{1\ 000\ \text{lb}}\right)$
1 and ½......	Aluminum	Aluminum	Aluminum	0.00035
1 and ½......	Steel	Steel	Steel	0.00012
1 and ½......	Steel	Aluminum	Steel	0.00014
1 and ½......	Steel	Aluminum	Aluminum	0.00030
1 and ½......	Aluminum	Aluminum	Steel	0.00017
¾ and ⅜......	Aluminum	Aluminum	Steel	0.00013
¾ and ⅜......	Steel	Aluminum	Steel	0.00010

Three large riveted specimens were tested, representing splices between a steel plate and one of aluminum. The specimens were 5¾ in. wide, using aluminum cover-plates and embodying five ¾-in. rivets on either side of the splice. The rivets were spaced on 4-in. centers. Details of the three specimens are given in Table 5.

The distribution of the load on the various rivets was determined by measuring the stresses existing in the cover-plates at sections between the rivets, the difference between the load on two adjacent sections being a measure of the load transmitted by the intervening rivet. Strains were measured with a 2-in. Berry strain-gauge on twenty-four gauge lines at each section between two adjacent rivets. The relatively low modulus of elasticity of aluminum permits the accurate determination of comparatively low stresses, the strain in aluminum being about three times as great as that produced by the same stress in steel. The joints were investigated for loads ranging to somewhat above the nominal design load of the specimen, Specimen No. 2, Table 5, being loaded in both tension and compression.

A summary of the results of the tests is shown in Table 5, together with the calculated values of the percentage of the load transmitted by each rivet for a load corresponding to a stress slightly less than the nominal working stress of the joints. The rivets are numbered from 1 to 5, No. 1 being the outer rivet on either end of a joint. Where the main plate and the cover-plates are of the same material and the latter is one-half the thickness of the former, the loads transmitted by the end rivets (Nos. 1 and 5) have been averaged, as have the loads for Rivets Nos. 2 and 4. In the case of the aluminum cover-plates on the steel main plate, however, each rivet transmits a different part of the load (because of the difference in the values of the modulus of elasticity), and the values for each rivet have been listed individually.

TABLE 5.—PERCENTAGE OF TOTAL LOAD CARRIED BY EACH RIVET IN A LARGE BUTT-JOINT

(At Nominal Working Load)

Specimen No.	Thickness of main plates, in inches	Thickness of cover-plates, in inches	Kind of rivets	Determination	ALUMINUM MAIN PLATE RIVETS NOS.			STEEL MAIN PLATE RIVETS NOS.				
					1 and 5	2 and 4	3	1	2	3	4	5
1......	$\frac{3}{4}$	$\frac{3}{8}$	Steel	Experimental..	34	13	6	17	8.5	12.5	27.5	34.5
				Calculated.....	31.5	14	9	17	9	10	19.5	44.5
2......	1	$\frac{1}{2}$	Steel	Experimental	29.5	14	13	23	14.5	16	22	24
					*25.5	19	11	16	17	16	19	32
					†25.5	18	13	18.5	9.5	13.5	23.5	35
				Calculated.....	28	16	12	17	12	13	20	38
3......	1	$\frac{1}{2}$	Aluminum	Experimental..	24.5	19	13	19.5	10	19	21.5	30
				Calculated.....	24.5	18	15	17	15	16	21	31

* Specimen straightened before test. † Specimen loaded in compression.

Considering the uncertainties that are introduced in the fabrication of such a joint, the results shown in Table 5 indicate a fair agreement between the calculated and measured values for the percentage of load transmitted by the various rivets. For the case of a steel main plate, aluminum cover-plates, and steel rivets, the calculated values indicate a more serious condition of over-load on the outer rivet than was measured experimentally.

Specimen No. 3 failed by shearing the rivets at a load of 151 500 lb, which corresponds to the calculated strength of the joint, calculated as the total shear strength of all the rivets. This fact suggests that at the breaking load each rivet was carrying an equal share of the load.

The frictional resistance of a riveted joint, as a factor in its behavior, is frequently given considerably more importance than it merits. According to the theory of static frictional resistance, load on the joint is resisted by frictional forces between the plates, and is accompanied by no relative movement of the adjacent faces of the plates. Mr. Hrennikoff discusses the action of the static friction in a joint having more than two rivets in a longitudinal row. His conclusion is correct that "a method of design based on allowing a uniform value for each rivet, working in friction [as is done in designing for shear], is wrong." However, the statement that only the outer rivets are effective in static friction, the intermediate rivets performing no work, is not entirely true. To maintain static friction, it is essential that there be no relative movement of the adjacent faces of the plates. This does not mean that the length of the plate, A_1B_1, must be the same as that of the plate, A_2B_2 (Fig. 1(*c*)). It is entirely possible for the two adjacent faces of the plates between A and B to remain the same length and yet for the shear detrusions to be distributed throughout the thickness of the plates so as to produce average changes of length that are different for the two plates. In the case of the lap-joint shown in Fig. 1(*c*), this phenomenon would be accompanied by bending of the plates, which is just what occurs when such a joint is subjected to a tensile load. It is possible, therefore, for the inner rivets to function in static friction before slip has occurred at the outer rivets. It is true that the percentage of the load that would be resisted by static friction of the inner rivets would be relatively small and, for practical purposes, the assumption that the outer rivets carry all the load (in static friction) is probably not far wrong.

Tests made on Specimen No. 2, Table 5 (which was fabricated with hot-driven steel rivets) revealed slipping in the joint even at very low loads. The results of these tests are shown in Fig. 13. Measurements were made of the change in length of the cover-plates between the first rivets on either side of the splice and over the gap between the two main plates. If the load were supported entirely by friction in the joint, the change in distance measured over the gap should be about the same as that measured in the

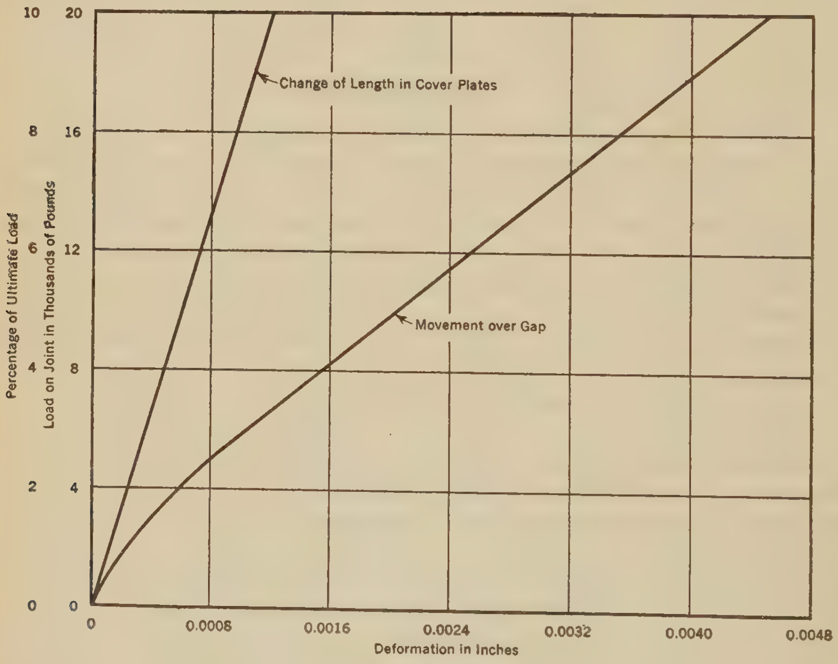

FIG. 13

cover-plates. Fig. 13 shows that the change in distance measured over the gap was relatively greater for extremely low loads, indicating a slip between the plates with the consequent loading of the rivets. Experience has shown that, in general, the friction between aluminum and aluminum, and between aluminum and steel (which are the conditions in Specimen No. 2, Table 5) is greater than the friction developed between steel and steel under the same circumstances. The frictional resistance in Specimen No. 2, therefore, was probably greater than would have been encountered in a similar all-steel joint.

From the results of the investigation, the following conclusions were drawn:

1.—At the working load, the load on a riveted joint is not equally distributed among the rivets in a longitudinal row, as is customarily assumed in design. In a double-strap butt-joint having cover-plates of one-half the thickness of the main plates, the end rivets on either half of the joint carry the greatest load, when the plates are of the same material. In the case of a steel main plate and aluminum cover-plates the first and second rivets from the splice are most highly stressed.

2.—At the breaking load, each rivet carries an equal share of the load, provided the rivets are the same size and of the same material.

3.—The problem of determining the distribution of the load among the various rivets of a joint such as those tested, can be solved mathematically, by considering the elastic deformations of the plates and assuming that the relative displacement between the plates at a rivet is proportional to the load transmitted by the rivet. The relationship between the relative displacement of the plates and the load on the rivet can best be determined experimentally.

4.—The manner in which the load is distributed among the various rivets in a joint depends on the following factors:

(*a*) The type and physical dimensions of the joint, including the width, thickness of plates, pitch, number, and diameter of rivets.
(*b*) The mechanical properties of the materials of the component parts.
(*c*) Uncertainties introduced by fabrication of the joint.

5.—Slip in the joints occurred at extremely low loads, indicating that the static frictional resistance of joints similar to those tested, is of minor importance in a consideration of its behavior under load.

While the unequal distribution of load on the rivets of a riveted joint is worthy of consideration, in the design of many structures with a relatively large factor of safety based on the ultimate strength of the structure, this consideration is not of primary importance. Since at the failure of a joint, the load is probably equally distributed among the rivets, such an assumption for design purposes seems rational. In certain special cases, however, such as riveted joints in machine parts, or structures that are subjected to a great number of repetitions or reversals of stress, the overload on the outer rivets may be of serious consequence, producing fatigue failures in the rivets. A rational design under such conditions should consider the unequal distribution of the load among the various rivets.

DONALD E. LARSON,[15] JUN. AM. SOC. C. E. (by letter).—An interesting analysis of the deformations and stresses in riveted joints is presented in this paper. Such an analysis reveals the fact that all the rivets in certain types of joints cannot possibly be stressed equally at working loads. In spite of this fact, nearly all riveted joints are designed in accordance with the assumption that the rivets are stressed alike, and this leads to a simple and convenient method of design. No design procedure, however, can be justified on the basis of simplicity if it does not provide adequately for the stresses to which the member is to be subjected. The real justification for proportioning joints on the basis of equally stressed rivets lies in the probability that, before failure occurs, the overstressed rivets will yield and thus will effect a more uniform distribution of load. It follows that the theoretical ultimate

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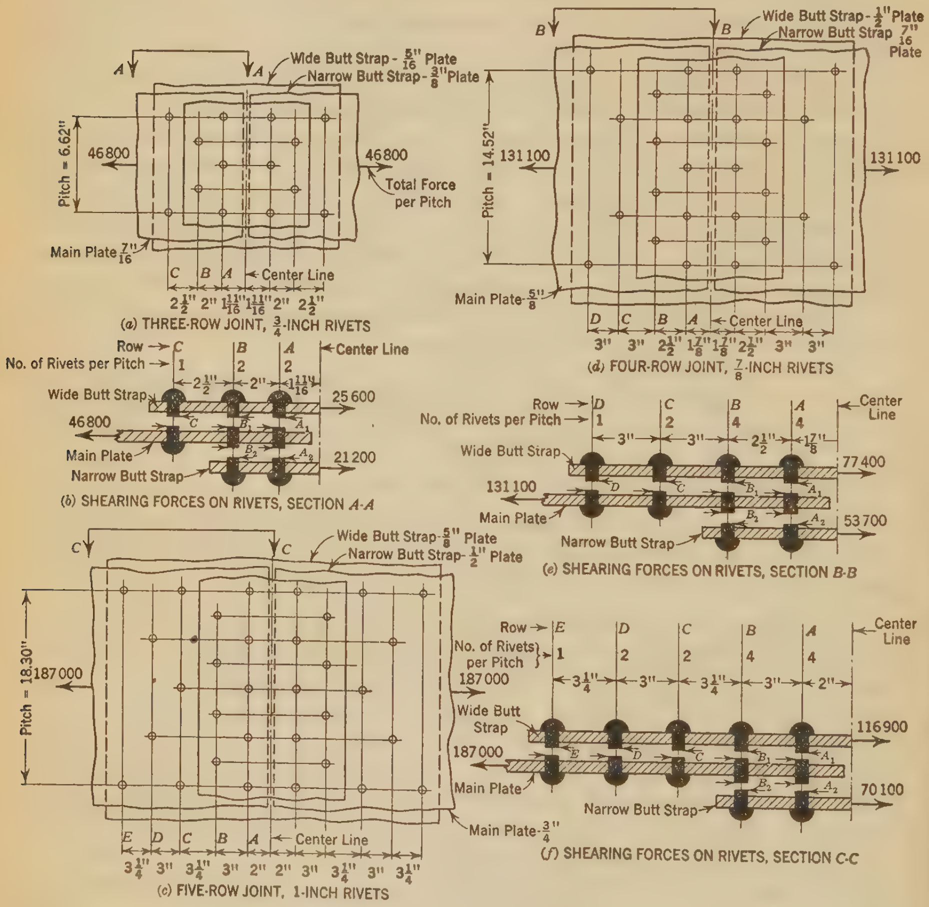
[15] Asst. Research Engr., Chicago Bridge & Iron Works, Chicago, Ill.


(a) THREE-ROW JOINT, $\frac{3}{4}$-INCH RIVETS

(b) SHEARING FORCES ON RIVETS, SECTION A-A

(c) FIVE-ROW JOINT, 1-INCH RIVETS

(d) FOUR-ROW JOINT, $\frac{7}{8}$-INCH RIVETS

(e) SHEARING FORCES ON RIVETS, SECTION B-B

(f) SHEARING FORCES ON RIVETS, SECTION C-C

A
A
Wide Butt Strap - 5/16" Plate
Narrow Butt Strap - 3/8" Plate
Pitch = 6.62"
46800
46800
Total Force per Pitch
Main Plate 7/16"
C B A Center Line
2½" 2" 1 11/16" 1 11/16" 2" 2½"
Row → C B A Center Line
No. of Rivets per Pitch → 1 2 2
2½" 2" 1 11/16"
Wide Butt Strap
25600
46800
C B1 A1
Main Plate
B2 A2
21200
Narrow Butt Strap
B
B
Wide Butt Strap - ½" Plate
Narrow Butt Strap 7/16" Plate
Pitch = 14.52"
131100
131100
Main Plate - 5/8"
D C B A Center Line
3" 3" 2½" 1 7/8" 1 7/8" 2½" 3" 3"
Row → D C B A Center Line
No. of Rivets per Pitch → 1 2 4 4
3" 3" 2½" 1 7/8"
Wide Butt Strap
77400
131100
D C B1 A1
Main Plate
B2 A2
53700
Narrow Butt Strap
C
C
Wide Butt Strap - 5/8" Plate
Narrow Butt Strap - ½" Plate
Pitch = 18.30"
187000
Main Plate - 3/4"
E D C B A Center Line
3¼" 3" 3¼" 3" 2" 2" 3" 3¼" 3" 3¼"
Row → E D C B A Center Line
No. of Rivets per Pitch 1 2 2 4 4
187000
3¼" 3" 3¼" 3" 2"
Wide Butt Strap
116900
187000
E D C B1 A1
Main Plate
B2 A2
70100
Narrow Butt Strap

FIG. 14.—STANDARD TYPES OF BUTT-JOINTS

strength of a joint designed on the basis of equally stressed rivets can only be attained when the rivets are made from a ductile material. If they were made of a material that obeyed Hooke's law up to the point of failure, as some materials do, it would be necessary to design all joints by some method similar to that outlined by the author. This design procedure would result in the selection of types of joints differing radically from those now in common use.

While it seems probable that the assumed condition of equally stressed rivets is actually attained before failure occurs, it is interesting nevertheless to speculate on the distribution of stress at working loads, and it is to be regretted that the author did not extend his study to include the more common types of joints used in the construction of large oil tanks, elevated water tanks, and pressure vessels. Standard butt-joints having three, four, and five rows of rivets are shown in Fig. 14. The rivet pitch shown for each of these joints is that which gives the maximum efficiency for the following assumed conditions: (1) All the rivets are equally stressed; (2) the effective diameter of each rivet is equal to the nominal diameter plus $\frac{1}{16}$ in.; (3) the value of the plate in tension = 1.0; (4) the value of the rivet material in shear = 0.75; and (5) the value of the rivet material in bearing = 1.5. Theoretically, the strength of the net section of the main plate is equal to the shearing strength of the rivets for each joint.

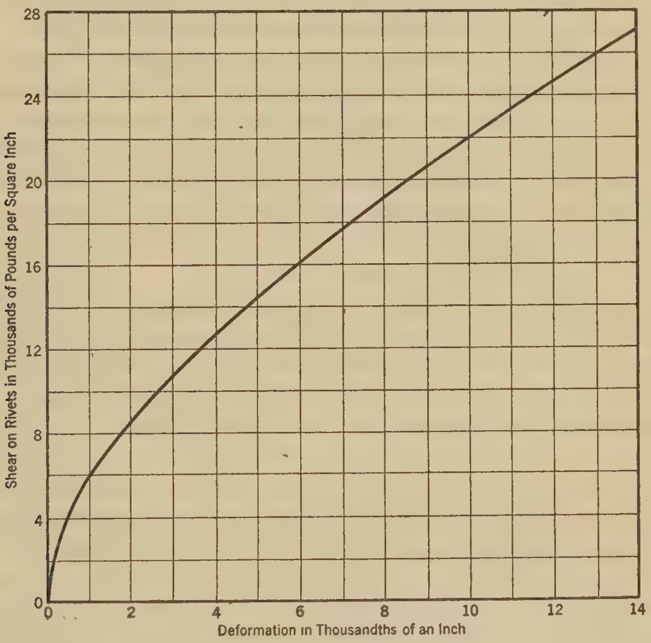


FIG. 15

In making an analysis of these joints to determine the stresses on the various rivets at working loads, the algebraic method used by the author was abandoned in favor of a method of successive approximation for the following reasons: (*a*) The algebraic expressions for the deformations in joints of this type are more or less cumbersome; and (*b*) the relation between the shear on a rivet and the deformation of the rivet is a variable quantity and is not a constant as assumed by the author. This relation between the unit shear and deformation is shown by the curve in Fig. 15. Although this curve is based on experimental data[16] obtained by measuring the slip in joints containing $\frac{5}{8}$-in. rivets, it has been used for determining the deformations of the larger rivets in the joints shown in Fig. 14, because of the lack of actual experimental data for these sizes. The rivet stresses are preferably determined as follows:

Step 1.—Apply a force to the main plate of the joint and, assuming that this force is equally distributed among the rivets, compute the tensile stresses in the main plate and in each of the butt-straps between successive rows of rivets.

Step 2.—Using stresses from Step 1 compute the elongation of the main plate and of each of the butt-straps between successive rows of rivets; then, from these elongations, compute the amount that each rivet must deform to preserve continuity.

Step 3.—Using rivet deformations from Step 2, determine from Fig. 15 the unit shear that must act on each rivet to produce the given deformation; and from these unit shears compute the total shear on each rivet.

Step 4.—Using the new set of rivet shears from Step 3, recompute the tensile stresses in the main plate and in each of the butt-straps between successive rows of rivets; then using this new set of stresses repeat Steps 2, 3, and 4 until the same results are obtained twice successively. When this condition is reached the total force acting on the joint is distributed among the rivets in such a manner that static equilibrium is maintained and the deformations of the rivets, the main plate, and the butt-straps are such that continuity is preserved in all parts of the joint.

The procedure outlined can be hastened considerably by guessing at the probable distribution of the force applied to the main plate in Step 1 instead of assuming that it is equally distributed among the rivets. Of course, the results obtained are exactly the same as those obtained by writing equations for the deformations and solving them simultaneously to determine the unknowns.

To each of the joints shown in Fig. 14, a force that will produce a unit shearing stress of 10 000 lb per sq in. on each of the rivets, if equally distributed among them as assumed in the design, has been applied. The actual distribution has then been computed by the four steps outlined, and the results are shown in Table 6. Each part of the table, 6(*a*), 6(*b*), and 6(*c*), shows the total shear acting on one rivet in each row, the unit shear on that rivet, and the percentage by which this unit shear differs from the 10 000-lb unit shear for which the rivet was designed. This analysis shows that for each of the three joints the rivets in the outer row are the ones most highly stressed. The stress on the outer rivet of the three-row joint is 14% greater than that for which it was designed, whereas the outer rivet of the four-row joint is 34% overstressed, and that of the five-row joint is 42% overstressed. The percentage of over-stress on the rivets in the outer row increases as the number of rows of rivets is increased.

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[16] See Fig. 39 in "Tests of Joints in Wide Plates," *Bulletin No. 239*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

Although it is evident from the foregoing analysis that the rivets in the outer rows of standard butt-joints are greatly overstressed at working loads, it can be stated definitely that this condition does not seriously reduce the ultimate strength of the joint, providing the rivets are made of a ductile material. This statement is based on experimental data obtained from tests[17] of joints in plates having a width of 6 ft. Two three-row joints having the exact dimensions shown in Fig. 14(*a*) and Fig. 14(*b*) developed an average ultimate strength equal to 98% of the theoretical strength computed from the control specimens, on the assumption that all the rivets were equally stressed. Both these test joints failed by tearing of the main plate and not by shearing of the rivets. Similarly, four, four-row joints having the dimensions shown in Fig. 14(*c*) and Fig. 14 (*d*) developed an average ultimate strength equal to 92.5% of the theoretical strength determined in the same manner. Two of these joints failed by shearing of the rivets and the other two failed by tearing of the main plate. These tests indicate that the actual ultimate strength attained by the joints are fairly consistent with the theoretical strengths computed by the usual method in which the rivets are assumed to be equally stressed. Although it would be better design practice to proportion riveted joints in such a manner that all rivets would be equally stressed at working loads as well as at the point of failure, the tests cited herein seem

TABLE 6.—ACTUAL DISTRIBUTION OF A FORCE PRODUCING A UNIT (DESIGN) SHEARING FORCE OF 10 000 POUNDS PER SQUARE INCH ON EACH RIVET

Item No.	Description	SHEARING FORCES (SEE FIG. 14)						
		A_1	A_2	B_1	B_2	C	D	E
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
((*a*) THREE-ROW JOINT, ¾-INCH RIVETS (SEE FIG. 14((*a*) AND FIG. 14(*b*))								
	Shear on One Rivet:							
1	Total, in pounds........	4 925	5 200	4 925	5 400	5 900	......	....
2	Unit, in pounds per square inch.................	9 480	10 000	9 480	10 400	11 400	......	....
3	Percentage difference from unit design shear on 10 000 lb per sq in	—5.2	0	—5.2	+4.0	+14.0	......	....
(*b*) FOUR-ROW JOINT, ⅞-INCH RIVETS (SEE FIG. 14(*c*) AND FIG. 14(*d*))								
	Shear on One Rivet:							
4	Total, in pounds........	6 700	6 825	6 500	6 600	7 700	9 200	....
5	Unit, in pounds per square inch.................	9 700	9 900	9 400	9 550	11 200	13 400	....
6	Percentage difference from unit design shear of 10 000 lb per sq. in.	—3.0	—1.0	—6.0	—5.5	+12.0	+34.0	....
(*c*) FIVE-ROW JOINT, 1-INCH RIVETS (SEE FIG. 14(*e*) AND FIG. 14(*f*))								
	Shear on One Rivet:							
7	Total, in pounds........	8 350	8 675	7 875	8 850	9 100	10 600	12 600
8	Unit, in pounds per square inch.................	9 400	9 730	8 820	9 950	10 200	11 900	14 200
9	Percentage of difference from unit design shear of 10 000 lb per sq in.	—6.0	—2.7	—11.8	—0.5	+2.0	+19.0	+42.0

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[17] "Tests of Joints in Wide Steel Plates," *Bulletin No. 239*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

to indicate that the usual method of design provides joints having the required ultimate strength, and, for this reason it does not seem likely that the present design method will be replaced by more exact methods in the near future.

A. E. R. DE JONGE,[18] M. AM. SOC. C. E. (by letter).—The problem dealt with by the author is not a new one. When such an old problem is discussed again one expects that either some new results are found, or some new and simpler method of dealing with it is established. The writer proposes to discuss how Mr. Hrennikoff's paper measures up to these two standards.

At an early date it was realized that the distribution of the load on a riveted joint over the various rivets is not uniform. This seems to have been clearly expressed for the first time as far back as 1867 by J. W. Schwedler,[19] in Germany, who unfortunately did not follow up this problem. Attention was once more drawn to this subject in 1881 by the late W. C. Unwin,[19a] Hon. M. Am. Soc. C. E., in England, and again, in 1884, by Th. Landsberg,[20] in Germany.

As far as the writer is aware, the first to state definitely that the outer rivets of lap-joints and the outermost and innermost rivets of double-butt strap joints transmit a greater part of the load than the intermediate rivets, was J. T. Milton,[21] in England (1885).

During some tests made by G. Lanza,[22] in 1887, at the Watertown, N. Y., Arsenal, of the United States Ordnance Department, an attempt was made to find the distribution of load on the various rivets from the uneven elongation of the rivet holes.

In 1892, C. Bach,[23] in Germany, showed from tests that the resistance of the rivets in the various rows was not the same, due to the unequal distribution of the force transmitted by the joint. In England, C. E. Stromeyer,[24] discussed, in 1893, the problem of the partition of load on the various rows of rivets, citing Milton's experiments.[21]

The same subject was discussed in the United States in 1902 by W. H. Boughton,[25] Assoc. M. Am. Soc. C. E., who investigated the question as to whether or not, in the light of the knowledge of an unequal distribution of the load over the various rivets, the usual rivet connections required changing. His answer was that no changes were required.

A discussion was started in the United States in 1904 by an anonymous letter in *Engineering News*,[26] on the same subject, in the course of which the Editor drew attention to the lack of systematic experiments on the subject.

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[18] With Babcock & Wilcox Co., New York, N. Y.

[19] *Wochenschrift* des Architekten Vereins zu Berlin, Vol. 1, pp. 451–452, November 22, 1867; pp. 463–464, November 29, 1867; and pp. 472–473, December 6, 1867.

[19a] *Proceedings*, Inst. of Mech. Engrs. (London), pp. 301–368.

[20] *Centralblatt der Bauverwaltung*, Vol. 4, May 17, 1884, pp. 201-203.

[21] *Transactions*, Inst. of Naval Archts. (London), Vol. 26 (March 27, 1885), pp. 204-210.

[22] Rept. of the Tests of Metals and Other Materials for Industrial Purposes Made at the Watertown Arsenal, 1887, pp. 882-923.

[23] *Zeitschrift* des Vereins Deutscher Ingenieure, Vol. 36 (October 1, 1892), pp. 1141-1148; and Vol. 36 (November 5, 1892), pp. 1305-1314.

[24] "Marine Boiler Management and Construction," by C. E. Stromeyer, Longmans Green & Co., (London), 1893.

[25] *Proceedings*, Ohio Soc. of Surv. and Civ. Engrs., 1902, pp. 17-26.

[26] *Engineering News*, June 9, 1904, pp. 542 *et seq.*

The first, to the writer's knowledge, to undertake a rigid mathematical analysis of the subject was Ivan Arnovļevič,[27] who in Austria, in 1908 and 1909, derived general formulas for the load transmitted by each rivet of a double-butt strap joint. While it is true that Arnovlevič only treated of butt-joints, his method is quite general and is equally applicable to lap-joints. He showed that the two outermost and the innermost rivets of each half of a butt-joint transmit most of the load, while the intermediate rivets transmit only a very small part of it. He further showed that on account of this distribution more than five rivets in line are useless, since the additional rivets transmit insignificant fractions of the load only. In addition, he dealt with bracket-plate and chord girder plate connections. In his conclusions, Arnovļevič definitely states the laws as revealed by his investigation and gives finally the following recommendations:

1.—The rivet pitch in the direction of the bar should be as small as possible.

2.—Fewer rivets of a large diameter are more advantageous than more rivets of a small diameter.

3.—More than five rivets in a line parallel to the axis of the bar are useless.

4.—Excessively strong butt straps impair the strength of the joint.

5.—The connection of truss members by means of gusset-plates is more advantageous than their connection to the web-plates of chord girders.

His solution is very elegant and quite general. It allows of the checking of the formulas by the determination of the "rivet factor" from experimental results, and thus his investigation opened the way for a rational treatment of riveted joints in general, which, however, was unfortunately never attempted.

Tests made in 1911 by Max Rudeloff,[28] in Germany, and reported on again later by F. Kögler[29] showed the unequal distribution of the load by the unequal elongation of the holes.

William H. Burr, M. Am. Soc. C. E., in the United States, likewise discussed[30] this problem. (See the Sixth (1904) and Seventh (1915) Editions of Professor Burr's book.)

In 1916, R. N. Blackburn[31] published in the United States a study on unsymmetrical riveted boiler joints and showed that these joints (lap-joints and butt-joints with a wider inner than outer butt strap) are less efficient than symmetrical double-butt strap joints due to the unequal distribution of the forces and stresses.

In 1916, Professor Cyril Batho,[32] of McGill University, Montreal, Que., Canada, made an exceedingly careful investigation of the partition of load in riveted joints and derived, by means of the principle of least work, theoretical formulas for the load transmitted by the various rivets. He checked his theoretical results by experiments in which the strain at various points of the joints was measured by means of very sensitive strain-gauges. The result of these experiments proved to be in good agreement with his theory. He thus proved that a riveted joint may be considered as a statically indeterminate structure and that the loads carried by each rivet may be obtained by the principle of least work in terms of a quantity, K, which depends on the manner in which the work is stored in the joint. He determined this coefficient, K, both theoretically and experimentally, finding fair agreement. He also considered the influence of an unequal distribution of the stresses in the plates, both transversely and longitudinally, the unequal distribution of the load between the two cover-plates, and the influence of the difference of the modulus of elasticity of the cover-plates to that of the main plate, as well as other types of connections.

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[27] *Oesterreichische Wochenschrift für den öffentlichen Baudienst*, Vol. 14 (August 22, 1908), pp. 607-615; also, *Zeitschrift für Architektur and Ingenieurwesen*, Vol. 50 (Old Series), Vol. 14 (New Series) (1909), pp. 89-106.

[28] Verhandlungen des Vereins zur Beförderung des Gewerbefleisses, Supplement to Vol. 90 (1911), pp. 1-82; see, also, the photograph given.

[29] Berichte des Auschusses für Versuche im Eisenbau, Edition B, No. 1, 1915.

[30] "The Elasticity and Resistance of the Materials of Engineering," by William H. Burr, John Wiley & Sons, N. Y., 1915.

[31] *Power*, Vol. 44 (August 8, 1916), pp. 203-206.

[32] *Journal*, Franklin Inst., Vol. 182 (November, 1916), pp. 553-604.

Professor Batho's results, although obtained in a totally different way, agree well with those of Arnovlevič.[27] Professor Batho also showed how to calculate the distribution of load in lap-joints and thus goes a step further than Arnovlevič. He covered, therefore, a great part of the field of riveted joints as used in structural engineering.

In Austria, Paul Fillunger[33] again treated of this subject in 1919, but considered the rivets as mere points being completely rigid while the plates only were subject to deformation. His results are somewhat similar to those of Arnovlevič[27] and Batho.[32]

In 1920, James Montgomerie,[34] in England, made a number of very careful experiments for the Committee of Lloyds Register, in order to ascertain the elastic behavior of riveted joints. He investigated lap-joints only, and the measurements with a very sensitive strain-gauge confirmed the fact that the distribution of both load and stresses was nowhere uniform throughout any of the joints.

In the same year, D. Rühl,[35] in Germany, made very careful measurements of the strains in a plate around the rivet holes and showed that the elongation is little influenced by the uneven distribution of stresses, a fact already shown by Batho.[32]

In 1920, Cl. Findeisen,[36] in Germany, also made experiments on the distribution of the stresses in the cover-plates of butt-jointed flat bars, using cylindrical well-fitting bolts as connectors. His investigations and careful measurements are of value in so far as they permit of the determination of the distribution of load over the various pins and the checking of certain constants, as explained later.

John S. Watts,[37] in the United States, pointed out in 1921 again, as did Mr. Blackburn,[31] that unsymmetrical double-butt strap joints with wider inner than outer butt strap (as used in boilers) are less efficient than symmetrical double-butt strap joints due to the better distribution of the load in the latter.

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[33] *Oesterreische Wochenschrift für den öffentlichen Baudienst,* 1919, Nos. 7-8.

[34] *Transactions,* Inst. of Engrs. and Shipbuilders in Scotland, Vol. 63 (March 30, 1920), pp. 284-312.

[35] *Forschungsarbeiten* des Vereins Deutscher Ingenieure, Heft 221, 1920.

[36] *Loc. cit.,* Heft 229, 1920.

[37] *Boiler Maker,* Vol. 21 (October, 1921), pp. 278-279.

In 1922, A. Hertwig,[38] in Germany, proposed using the deflection of the rivet heads as a means of determining the distribution of load on the various rivets.

R. Maillard,[39] in Switzerland, dealt in 1923 with the same problem and recommended short joints, rather broader than those used at present, or joints with plates tapered or stepped in the measure in which the forces transmitted decrease, or increase, respectively.

James Montgomerie,[40] in England, supplemented in 1923 his former experiments for Lloyds Committee[34] by tests on lap-joints of heavier plates and larger rivet diameters. His results were similar to those obtained by him for his first series of experiments.

In 1923, Friedrich Bleich[41] published in Germany an investigation on the distribution of the stresses in a bar of rectangular cross-section. His study is remarkable in that it is not based, as were all the others, on the usual simplified Navier theory of stress distribution, but on the much more general theory of elasticity for a two-dimensional state of stress, a state to which riveted joints closely conform. He investigated the problem of stress distribution due to local loads, such as are produced by the rivets on a plate, and *vice versa*, and derived, by means of the Airy stress function, formulas for the distribution of the longitudinal stresses in riveted joints.

In the following year (1924) Bleich published another theory.[42] Due to his former study being based on theoretical deductions only, he based his new theory on the work of Arnovļevič,[27] but treated the subject in a more practical manner by introducing for the movement of the plates at the rivets a proportionality factor which he calls the "slip modulus," assuming that the displacements are proportional to the loads transmitted by the rivets. He determined this slip modulus from the experiment by Findeisen[36] for pins, and for rivets from tests by Rudeloff,[23] introducing the value thus directly obtained from experiments, into his formulas. By means of this slip modulus, he found the percentages of the load transmitted by the various rivets, which percentages, however, are nowhere near those determined by either Arnovļevič or Batho,[32] and indicate a much more even distribution of load (his values being only a small percentage greater than those determined from the theory of the uniform load distribution). The slip modulus is the reciprocal value of Mr. Hrennikoff's constant, k. Thus, Bleich has already done what the author now recommends in his paper, in that he actually determined the all important constant, k, from experimental data, in order to arrive at more correct results.

The Chief Engineer for the Swiss Association of Boiler Proprietors, E. Höhn,[43] carried out in Switzerland, in 1925, an exceedingly careful series of experiments on the behavior of riveted joints. Höhn measured the relative displacements between the bars and butt straps of double-butt strap joints for thin as well as for thick plates at various points of the joint. The stress distribution obtained therefrom was found to be nowhere uniform either in the transverse or longitudinal direction of the bar. The strain curves plotted by Mr. Hrennikoff are somewhat similar to those plotted by Höhn, but Höhn's curves are by no means straight lines. Höhn discussed his important results at great length and gave also rules regarding the lay-out of riveted joints.

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[38] *Der Bauingenieur,* Vol. 3 (March 21, 1922), p. 170.

[39] *Schweizerische Bauzeitung,* Vol. 82 (July 28, 1923), pp. 43-45.

[40] *Transactions,* Inst. of Naval Archts. (London), Vol. 65 (March 23, 1923), pp. 179-197.

[41] *Der Bauingenieur,* Vol. 4 (May 15, 1923), pp. 225-299; May 31, 1923, pp. 304-307; and June 15, 1923, pp. 327-331.

[42] "Theorie und Berechnung der eisernen Brücken," Berlin, Julius Springer, 1925.

[43] "Nieten und Schweissen der Dampfkessel," Berlin, Julius Springer, 1925.

A further remarkable series of experiments was carried out in 1926 by Commander E. L. Gayhart, U. S. N. Some of the results were published by him and Professor William Hovgaard.[44] This series of experiments is of the utmost importance, not only because they were carried out with exceptionally large double-butt strap joints, but also on account of the great accuracy of the measurements. The results confirmed the findings of the previous experimenters and investigators in that they show for all stages of loading (just as in Batho's case[32]) that the outer rivets carry by far the greater amount of load, although the inner rivets take an increasing share of it as the load increases. Unfortunately, Commander Gayhart, contrary to Batho, has not published the necessary data of the strain measurement which would permit of a check. It is, therefore, not possible to derive, from the results of Commander Gayhart, the value of the constant, *k*, of Mr. Hrennikoff.

A. Hertwig and H. Petermann[45] in Germany attempted, in 1929, to solve the same problem by the method suggested by Hertwig,[38] which consisted in measuring, by optical means, the angular deflections of the ends of the rivet heads due to the bending of the rivets. This attempt was unsuccessful, however, and the experimenters substituted well fitting pins for the rivets just as were used by Rühl[35] and Findeisen.[36] The results thus obtained again showed that the distribution of the load is not uniform. As a check the second of the investigators derived general formulas for this case.

How important the determination of the load distribution in riveted joints is held in England, may be seen from the First Report of the Steel Structure Research Committee of the Department of Scientific and Industrial Research of the British Government,[46] published in 1931. Professor Batho's method[32] of calculating the load distribution in riveted joints was incorporated in this report.

Otto Graf in Germany published[47] in 1931 a booklet in which he reported on experiments with riveted and welded bars of various steels under high sustained loads (long-time strength tests) and under fluctuating tension loads. He found that the bearing stresses at the holes changed the state of stress distribution in the main bars so that fracture under repeated loading took place ahead of the outermost rivets of the joint and not at the weakest cross-section in the first rivet row as in ordinary tension tests. He also carried out investigations into the load distribution under fluctuating tensile loads. While these tests were not completed at the time the booklet was published, the results obtained to that time indicated that the distribution of load under often repeated load variations is much more uniform than was to be expected from the older investigations with pins (obviously, Findeisen's investigation[36] and those by Hertwig and Petermann[45] are meant), which latter investigations were carried out as ordinary tensile tests. The increase in temperature of the rivet heads in joints with three rivets in line differed only slightly, thus showing a more even distribution under varying loads.

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[44] *Transactions*, Soc. of Naval Archts. and Marine Engrs., Vol. 34 (November 11, 1926), pp.. 55–74.

[45] *Der Stahlbau*, Vol. 2 (December 13, 1929), pp. 289-298.

[46] H. M. Stationery Office (Lond.), 1931, pp. 100-179.

[47] "Dauerfestigkeit von Stählen mit Walzhaut ohne und mit Bohrung von Niet-und Schweissverbindungen," by O. Graf, pub. by the V. D. I.-Verlag, Berlin, 1931.

The results of these tests were published[48] in 1932. The author discussed the importance of the frictional resistance and referred to Wellinger's "Dissertation"[49] in which the means of obtaining the highest possible clamping forces with rivets were described. Not until the frictional resistance is overcome, do the rivets touch the walls of the rivet holes.

Graf discusses here in greater detail than in his booklet the fact that, under the influence of friction and repeated loading, fracture occurred in the plate slightly ahead of the first rivet. He also showed that the relative displacements of the bars are permanent to a large extent, that a varying load is distributed more evenly than a mere statical load, and that the total deformations were greater than for ordinary tensile tests, the permanent part having increased while the elastic part was altered but slightly.

By investigating the deformation and final destruction of rivets under often repeated loads, Graf made a further considerable step forward. The extent to which the forces in the joint are transmitted by the various rivets coming into direct contact with the wall of the holes, after overcoming the frictional resistance, can be derived from the deformation of the rivets. These were studied by R. Wörnle[49] who, under Graf's guidance, measured the deflection by means of small pins having three collars and being fitted tightly into holes drilled longitudinally through the rivets. The bending of these pins was measured by means of mirrors, and the load transmitted by each rivet was determined therefrom. In this way Graf found that in riveted joints with high rivet clamping forces a very much smaller part of the load is transmitted by the direct action of the rivets than is obtained from the usual calculations which neglect friction. Further, the differences in the load transmitted, under the condition of Graf's experiments, by three or more rivets in line, were less than those in the case where no frictional resistance is assumed to be present.

Attention should further be drawn here to the painstaking investigations of Th. Wyss,[50] in Germany, who in 1923 derived from innumerable strain measurements the distribution of stress in riveted connections, for example, in the gusset-plates connecting members to a chord girder; and, further, to the investigations of Findeisen,[51] in Germany, and Stefan Gállik,[52] in Austria, both published in 1928, who determined the bearing pressure between rivets and plates.

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[48] *Zeitschrift* des Vereins Deutscher Ingenieure, Vol. 76, April 30, 1932, pp. 438-442.

[49] Dissertations pub. by the Technical Coll. at Stuttgart, 1931.

[50] *Forschungsarbeiten* des Vereins Deutscher Ingenieure, Heft 262, 1923.

[51] Rept., Second Inter. Cong. for Bridges and Structural Eng., Vienna, 1928, pp. 347-364.

[52] *Loc. cit.,* pp. 365-402.

After thus having given a bird's-eye view of the most important research carried out prior to the publication of Mr. Hrennikoff's paper, and after having clearly shown that this problem is not a new one, it is pertinent to investigate the second question, as to whether Mr. Hrennikoff's paper brings a new or simpler method for the determination of the load distribution.

The title of the author's paper leads one to believe that he intends to investigate the work of riveted joints, that is, the storage of energy in riveted joints and the behavior of the joint during such storing of energy; but, in actual fact, all he undertakes is an investigation into the static distribution of load over the rivets and, therefore, it would be more appropriate if he had referred to "the behavior of riveted joints" or "the load distribution in riveted joints."

Mr. Hrennikoff has chosen for his analysis a "logical" rather than a rigid mathematical treatment. In general, he pictures the phenomena occurring in a riveted joint correctly. It is difficult, however, to agree with his statement that "designing on the basis of friction seems to be quite correct in the case of only one rivet in a longitudinal row * * *." (Mr. Hrennikoff uses "longitudinal row" instead of the accepted terminology which designates rivets in the direction of the transmitted force as rivets "in line" and those transverse to it as "rivets in rows" or "rivet rows"). Even if there were but one rivet, slip at the faying surfaces must occur, because the force, which is transmitted by the rivet due to friction, at the rivet heads and between the plates, is not transmitted at one single point only, but is spread over a fairly wide area. Therefore, it is gradually transmitted from one plate to the adjoining one, thus causing in both plates elongations and distortions which differ according to the magnitude of the force that has already been transmitted up to the point under consideration.

His further statement that "it is reasonable to assume that the other part, X_{AS}, of the total force, X_A, developed by the rivet (that due to shear), is proportional to the slip of plates at Point *A*," forms the basis of his investigation and, therefore, should be given due prominence. His assumption, however, is no more than a first approximation since the plates do not deform in the longitudinal direction only, but also in the transverse direction, particularly around the rivet holes. On account of this deformation the slip changes over a wide area, and it is impossible to state, the slip of which point should be regarded as "the slip." Thus, Mr. Hrennikoff can undoubtedly refer to the "mean slip" only and thus arrive also at a first approximation only. Arnovļevič[27] and Bleich,[41] who have done exactly the same, have not neglected, however, to state so expressly. Furthermore, the author has completely neglected friction.

Since it is doubtful whether a rigid analysis of the entire problem by means of the theory of elasticity will ever be possible, the writer agrees that some such simplifying assumption is necessary; but a suitable factor should be included allowing for the adaptation of the results of the mathematical analysis to the results from experiments. Mr. Hrennikoff, like Arnovļevič and Bleich, has incorporated this factor in his rivet factor, *k*.

The derivation of his formulas follows the "logic of Part I," but it must be stated that his "logical" method rather clouds the issue. A plain mathematical analysis accompanied by a simple diagram showing the deformations would have obviated a large part of the complicated description and would have been more to the point in that the main formulas could have been derived directly and would have led to general formulas instead of to specific ones. Besides, it would also have obviated the omission of the deductions for cases having more than three rivets and for butt-joints.

If one analyzes mathematically what lies behind the long explanations, one finds that Mr. Hrennikoff compares the respective motions of the two plates at the various rivets with one another. For this purpose he chooses as "a fixed point in space" the center line of the rivet at the extreme right-hand end of the upper plate; but the lower plate at this very point still moves to the right for a distance $-l$.

All this would have been unnecessary if he had drawn a diagram, such as Fig. 16. It would have been evident, therefrom, that the only requirement is to equate the distances of the respective center lines from any rivet head at one plate to the next following rivet head at the other plate, using one time the deformations of one plate and rivet and the other time those of the other plate and rivet. This, after all, is exactly what Mr. Hrennikoff has done without expressly stating it. In this manner, he would have obtained the following system of equations, using the notations of Fig. 16:

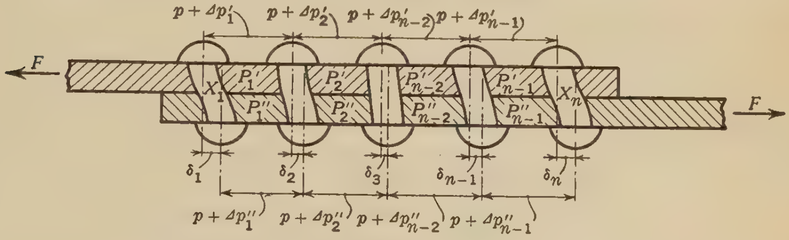

FIG 16

$$p + \Delta p'_1 + \delta_2 = p + \Delta p''_1 + \delta_1 \qquad \text{(28)}$$

or,

$$\left.\begin{array}{l} \Delta p'_1 + \delta_2 = \Delta p''_1 + \delta_1 \\ \Delta p'_2 + \delta_3 = \Delta p''_2 + \delta_2 \\ \Delta p'_3 + \delta_4 = \Delta p''_3 + \delta_3 \\ \cdots\cdots\cdots\cdots \\ \Delta p'_{n-1} + \delta_n = \Delta p''_{n-1} + \delta_{n-1} \end{array}\right\} \qquad \text{(29)}$$

The respective elongations, $\Delta p'_1$, $\Delta p''_1$, etc., as well as the deflections of the rivets, $\delta_1, \delta_2, \ldots, \delta_n$, can then be expressed by the forces acting in the various plate sections and by the forces transmitted by the various rivets. If the forces transmitted by the various rivets are $X_1, X_2, X_3, \ldots\ldots, X_n$, and the forces acting in the plates between rivets, m and $m + 1$, are in the upper plate, P'_m, and in the lower plate, P''_m, then there exists the following system of equations (designating the load on the joint by F):

$$\left.\begin{array}{l} F - P'_1 = X_1 \\ P'_1 - P'_2 = X_2 \\ P'_2 - P'_3 = X_3 \\ \dots\dots\dots\dots\dots \\ P_{n-1} = X_n \end{array}\right\} \dots\dots\dots\dots\dots\dots\dots\dots (30)$$

and, by addition,

$$X_1 + X_2 + \dots + X_n = F \dots\dots\dots\dots\dots\dots\dots\dots (31)$$

It should be emphasized that Equation (31) is not an independent one. Furthermore,

$$\left.\begin{array}{l} P'_1 + P''_1 = F \\ P'_2 + P''_2 = F \\ \dots\dots\dots\dots\dots \\ P'_{n-1} + P''_{n-1} = F \end{array}\right\} \dots\dots\dots\dots\dots\dots\dots\dots (32)$$

Since,

$$\left.\begin{array}{l} \Delta p'_1 = \dfrac{P'_1 p}{E w t_1}; \ \Delta p''_1 = \dfrac{P''_1 p}{E w t_2} \\ \Delta p'_2 = \dfrac{P'_2 p}{E w t_1}; \ \Delta p''_2 = \dfrac{P''_2 p}{E w t_2} \\ \dots\dots\dots\dots\dots\dots\dots\dots\dots \\ \Delta p'_{n-1} = \dfrac{P'_{n-1} p}{E w t_1}; \ \Delta p''_{n-1} = \dfrac{P''_{n-1} p}{E w t_2} \end{array}\right\} \dots\dots\dots\dots (33)$$

and, since,

$$\left.\begin{array}{l} \delta_1 = kX_1 \\ \delta_2 = kX_2 \\ \dots\dots\dots \\ \delta_n = kX_n \end{array}\right\} \dots\dots\dots\dots\dots\dots\dots\dots (34)$$

there are six systems of equations, or $6n$-4 equations in all, for the $6n$-4 unknowns—from which the $6n$-4 unknowns (and, consequently, the unknown rivet forces, X_1, X_2, ..., X_n), can be determined. In this way general formulas may be derived which yield for the cases dealt with by Mr. Hrennikoff exactly the same formulas as found by him. It would be very interesting to know, exactly how he has derived the formulas for more than three rivets, because he gives the derivation of this simplest case only; but by whatever mathematical or "logical" process he has arrived at these solutions, the general method is exactly the same as that used by both Arnovlevič[27] and Bleich.[41]

The most important point in the entire evaluation of the formulas, however, is the determination of the factor, *k*. It is creditable that Mr. Hrennikoff, like Arnovļevič, has attempted to determine this factor analytically. Unfortunately, Mr. Hrennikoff has not given this derivation so that it is impossible to determine the accuracy of his results. Mr. Hrennikoff obviously feels, and rightly so, that this determination can only be regarded as a first step and advocates the determination of *k* by experiment. Bleich has previously done this by attempting to determine this value, or rather its reciprocal value, from Rudeloff's experiments.[26] Obviously, Bleich did not know of Batho's anaylsis and experimental data,[32] because he could have used these to check his value for the "slip modulus." Thus, Bleich went a step further than Mr. Hrennikoff, and the latter's results can be of no more than academic interest.

The writer would like to give Mr. Hrennikoff credit for one advance that his analysis has established. He has resolved the cross-sectional area of the plates and cover-plates into width and thickness. Thus, he is able to get, for the same plate width, from his formulas directly the rivet coefficient for any plate thickness while the formulas of his predecessors do not allow of this directly, requiring a further though short calculation.

In valuing Mr. Hrennikoff's paper it must be pointed out that he has given a solution for a limited number of cases only; that is, for lap-joints containing up to six rivets and for butt-joints containing up to three rivets. His predecessors, however, have given general formulas which they evaluated for various cases, for double-butt strap joints up to six rivets. If one further compares Mr. Hrennikoff's four conclusions with those given by his predecessors, it is difficult to find anything new in them.

It may not be amiss to state further, that Mr. Hrennikoff has not taken into consideration the bending in lap-joints due to the moment created by the plates not lying in one plane. This changes the distribution of forces materially on account of the bending it induces in the rivets in addition to that present if the plates were not subjected to this bending moment.

At the beginning of Part I, Mr. Hrennikoff makes the statement that riveted joints "are not well understood." To this very true statement the writer would like to add that "the literature on riveted joints" does not seem to be well known, either. This has been the reason why he has prepared[53] a rather comprehensive "Bibliography on Riveted Joints." He would recommend its careful study to all who intend to carry out any experimental or theoretical work on riveted joints. It is simply amazing how much has been duplicated, how many immensely costly experiments have been carried out, the readings and data of which have never been published and, therefore, cannot be evaluated to the limit of their usefulness. Any progress in riveted joints can only come from a careful study of all the results obtained thus far and from outlining further tests with the idea of checking up experimentally the factors involved, a procedure which has also been recommended by Mr. Hrennikoff.

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[53] "Riveted Joints: A Historical Survey of Their Development, with a Bibliography and Abstracts from the Literature," by A. E. R. de Jonge, *Research Publications*, A. S. M. E. (1934).

The writer would like to take this opportunity to sound a note of warning with regard to the calculation of riveted joints based on insufficient assumptions. As almost no structure is subjected to a steady direct load, but rather always to variations in load, these latter should receive due attention. Further, it is useless to try building up a rational theory of riveted joints without taking the frictional resistance of the joint into consideration. Only by the co-ordination of all factors having a bearing on the resistance of riveted joints can a theory be arrived at which will describe the behavior of riveted joints more or less correctly. Graf's investigations[47] [48] form a first step toward this end. If Mr. Hrennikoff's paper will have the effect of starting a new movement in the direction outlined by the writer, the latter can do no better than to thank the author heartily for his paper.

A. Hrennikoff,[54] Esq. (by letter).—The response given this paper and the points of interest and importance brought out in the course of the discussion are gratifying. In answering these points it will be necessary to reexamine some of the original statements in the light of new information.

The main claim was that, in a riveted lap-joint or butt-joint with more than two rivets in a longitudinal row, under working conditions, distribution of the load among the rivets would be non-uniform, the outer rivets taking the greater part of the stress. Most of the discussers accept this assertion, and some of them mention experimental evidence in its support. However, correct theoretical estimation of this inequality and of the relative values of factors instrumental in producing it is almost impossible. The difficulty lies in the uncertainty of the relation between the slip of the plates at the rivet and the force developed by it. In a joint under a steady direct load this relation is determined mostly by the following three factors: The resistance of rivet in shear and bearing; the friction force; and the clearance around the hole caused by cooling of the rivet. Diversity of conditions and methods of fabrication are chiefly responsible for the uncertainty in relative importance of these factors in any particular joint; therefore, the force-slip curves assume a variety of shapes.

Messrs. Hill and Holt, referring to their experiments with double-strap butt-joints of aluminum and steel plates, state that slip begins very early, that friction is comparatively unimportant, and that the force-slip curve does not depart from a straight line passing through the origin sufficiently to invalidate the results based on the straight-line relation.

The force-slip curve to which Mr. Larson refers (Fig. 15) is not straight. It begins vertically at the origin, indicating that very small loads are resisted by the rivet almost without any deformation, and then gradually diminishes its slope with the horizontal.

In the paper[55] from which Fig. 15 was taken, several other force-slip curves of entirely different character are given, of which Fig. 17 is typical. This curve has a long, nearly vertical section, which changes its slope rather sharply into one not far from horizontal. Although the ordinates of this curve represent the average shear stresses for all four rivets in the joint, and not for the rivet under consideration, a moment of thought will convince one that if it was possible to reduce the ordinates to shear stresses of the rivet in question, the general shape of the curve would still be preserved.

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[54] Instr., Dept. of Civ. Eng., Univ. of British Columbia, Vancouver, B. C., Canada.

[55] See Figs. 42, 45, and 48, in "Tests of Joints in Wide Plates," *Bulletin No. 239*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

These are the three types of the force-slip curves that have come to the writer's notice, and he believes that most of the rivets in riveted joints, if tested, would exhibit one of these shapes or some intermediate form. (It may be mentioned, by the way, that Mr. Roop's Fig. 12, representing a relation between force and over-all deformation of the joint, cannot be used here, because it is constructed on an entirely different basis.) The formulas in the paper are based on the first type, a straight line through the origin, indicating the absence of both friction force and clearance around the rivet, although the rivet deformation may not necessarily be entirely elastic. This is an extreme case, but an approach to it seems possible under certain conditions, as is illustrated by the discussion of Messrs. Hill and Holt, who have found fair agreement between the experimental and the theoretical (based on an experimental value of the rivet coefficient) stress distributions among the rivets, as shown in Table 5.

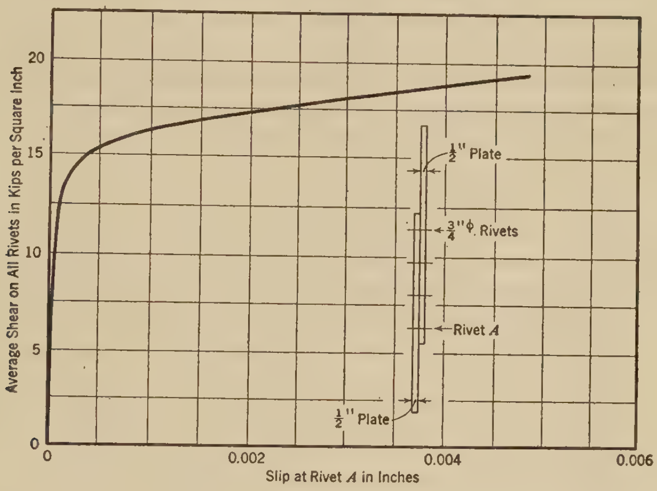

FIG. 17.

When the force-slip curve deviates considerably from a straight line these formulas naturally do not apply; yet, if the expected force-slip curve is known, or at least the limits between which it may fluctuate, the distribution of the load among the rivets may nevertheless be obtained theoretically by the excellent method of successive approximations suggested by Mr. Larson. The writer has applied the method to a few examples and has found that successive values of the rivet forces converge rapidly, so that the final results may be obtained after the second or third trial.

An analysis of a joint in which the rivets show the force-slip relation of the type represented in Fig. 15, still indicates an over-stress of the outer rivets. By using the curve of Fig. 17 the result may be different, and it seems appropriate to investigate this case in the light of the discussion presented by Mr. Shearwood, who strikes a rather different note, demonstrating how a friction force of a certain magnitude, combined with sufficient clearance around the rivet will cause equal stress distribution. The ideal stress-slip relation, according to Mr. Shearwood's assumption, seems to be as typified by Fig. 18. The rivet develops a force to the limit of its frictional resistance, almost without any deformation, after which it slips until all the clearance is absorbed, and the rivet shank comes in contact with the plates; then the rivet force increases again, owing to resistance in shear and bearing. The values computed by Mr. Shearwood are only illustrative, of course, but they are fairly representative of results to be expected under average conditions of fabrication. The resemblance which the curve of Fig. 18, from the origin to Point *A*, bears to the experimental curve, Fig. 17, is significant and testifies for the soundness of Mr. Shearwood's arguments. Since in this case a comparatively severe slip of the outside rivets requires no increase in rivet resistance, as long as any clearance is still left, Mr. Shearwood succeeds in demonstrating, by two examples of lap-joint and butt-joint, that if the working load is equal to the sum of the frictional resistances of all the rivets, the distribution of the load among the rivets will be perfectly uniform.

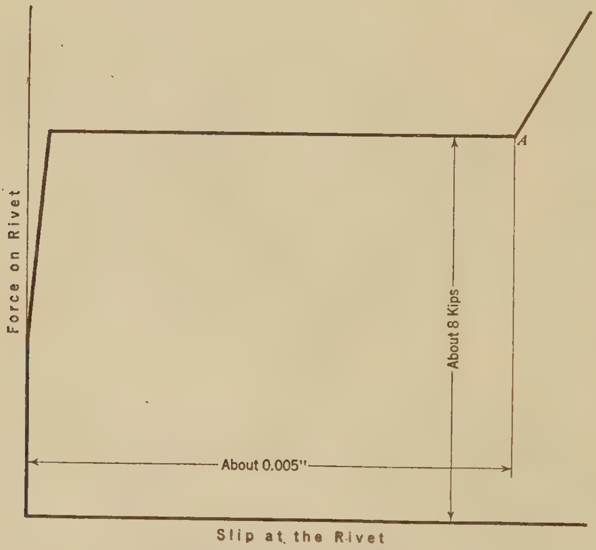

Fig. 18.—Force-Slip Diagram for ⅞-Inch Single-Shear Rivet.

If it happens that the working load is less than the sum of the frictional resistances of all the rivets, a certain number of inner rivets are idle, the remainder exerting forces equal to their frictional resistances. On the other hand, if the working load is greater than the sum of the frictional resistances of the rivets the plates slip one over the other, and the excess of the load is borne almost completely by the extreme outer rivets.

Thus, summarizing the results for various kinds of force-slip relations, an approximate equality of rivet forces under working conditions seems possible only for one type of the force-slip curve (Fig. 17) and for one definite relation between the load and the frictional resistance, over which the designer has no control. For other conditions the working load does not divide equally, and the outer rivets are overstressed.

Referring to the diagrams constructed by Mr. Troelsch in which he compares the results obtained by his weld theory with those of the rivet theory, one would not fail to notice their close agreement. Of course, as Mr. Troelsch explains, this agreement is only natural, since the underlying assumptions of both theories—proportionality between the stress and deformation in weld and rivet—are identical. It is only fair to state that Mr. Troelsch's weld formula,[56] independent as it is of the number of rivets, if they are spaced uniformly, is preferable to the rivet formulas, which increase considerably in complexity as the number of rivets is increased.

Mr. Roop presents an interesting discussion elucidating the behavior of riveted joints under stress. Assuming a continuous connection between the plates, he arrives at a formula for the stress in the plates, σ, which is reducible to Mr. Troelsch's Equation (23).[57]

There is no doubt as to the correctness of Mr. Roop's statement that the clamping effect of a rivet extends over a large area around it; on the other hand, one cannot see any mechanism through the medium of which continuous stress over all the faying surface would assume the intensity at any point proportional to the relative slip at the point. The friction force caused by clamping is, naturally, independent of slip, and is indeed ruinous to a theory based on proportionality between force and deformation. That is why, in developing his formulas, the writer (by assumption) eliminated friction from consideration and left all load to be resisted by the isolated units (the rivets) which presumably may develop forces proportional to deformations. The writer is thus more inclined to prefer the rivet-stress formulas based on continuous connection between the plates because of their greater simplicity for long joints and on their independence of the number of rivets, rather than because of closer conformity to reality.

The assumption of rivets of very small size ("point rivets"), attributed by Mr. Roop to the writer, appears not absolutely necessary, although the finite size of the rivets naturally requires some reduction in the width of the plate, or in the pitch, similar to that suggested by the writer, in order to take account of the rivet hole; and also, as Mr. de Jonge points out, "the slip" at the rivet becomes actually the average slip, since different points in the vicinity of the rivet slip somewhat unequally. Mr. Roop's discussion of the experimental phase of the question presents a valuable addition to his theoretical analysis.

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[56] See p. 413, Equation (15).

[57] See p. 414.

To Mr. de Jonge is accorded the credit of contributing an extensive bibliography on the subject. No doubt it will be of great value to future investigators, especially to those who are acquainted with German. The writer wishes to thank Mr. de Jonge for calling attention to two assumptions used in the paper, but which were not stated: One concerning the meaning of the word, slip; and the other relating to the effect of disregarding the bending of lap-joints, owing to their eccentricity. (The latter has also been noted by other discussers.)

Mr. de Jonge's way of attacking the problem of stress distribution among the rivets does not appeal to the writer. Introducing admittedly no new method, he uses an excessive number of unknowns; thus, $(6 \times 3 - 4) = 14$ equations would be required for the solution of a three-rivet lap-joint, where the writer uses only two equations. This economy in the number of unknowns is largely due (aside from the assumption of symmetry) to the use of the displacement diagram, Fig. 3, which assists in forming a mental picture of the deformations associated with the stressing of the joint. Answering a question raised by Mr. de Jonge, constructing such a diagram and equating the vertical distances between two of its curves to the slips of respective rivets, form the basis for the solution of other riveted joints.

The writer feels uncertain also as to the possibility of obtaining from Mr. de Jonge's equations a general solution, applicable to any number of rivets, unless such a solution would be so formidable as to be practically unworkable.

The justice of the main criticism advanced by Mr. de Jonge, that neither the subject-matter of the paper nor its method are entirely novel, must be partly conceded in view of the bibliographic data supplied by him; on the other hand, Mr. de Jonge joins the writer in his belief that the riveted joints "are not well understood," and this seems to be sufficient reason for the presentation of the present paper before the Engineering Profession. It is a significant fact in this connection that, according to Mr. de Jonge, only one theoretical investigation on the subject, that by Professor Batho,[32] using a different method, had been published in English previous to this paper.

A brief discussion of Professor Batho's work[58] with the view of co-relating its results with those of the writer seems to be in order. In the theoretical part of his paper, Professor Batho uses the method of least work; and the algebraic expression for the work of deformation of a rivet in terms of the rivet force, X, is assumed to be proportional to KX^2, in which, K is a coefficient determined by experiment, and is constant for all rivets of the joint under a given load. The form of the rivet-work function, quadratic in X, involves a tacit assumption of proportionality between the rivet force and its deformation (which is equal to the slip of the plates). Under these circumstances, the resulting expressions for the rivet forces are exactly equivalent to the expressions derived by the writer.

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[58] *Journal*, Franklin Inst., Vol. 182 (November, 1916).

In his experimental determination of the factor, K, for the double-strap butt-joints, Professor Batho does not use the force-slip relation for rivets; his method of attack is to measure the strains in cover-plates between different rivets. This enables him to prove that the outer rivets are overstressed, and to find experimentally the actual distribution of the load among the rivets. In conjunction with his theoretical formulas this leads to an experimental value of the coefficient, K, as follows: $K = \frac{F}{100\,A}$, in which, F is the total force on the joint, in kips; and A is the cross-section area of the upset rivet, in square inches. This formula is intended to be applied only to the joints with dimensions used by the investigator.

The fact that F appears in the expression for K seems to contradict the assumed independence of K from the rivet forces, X, the sum of which is equal to F. Probably Professor Batho's result is due to the shape of the actual force-slip curves which in his tests apparently deviated from the tacitly assumed straight lines, in a manner similar to Fig. 15.

The theoretical relation between the writer's rivet factor, k, and the coefficient, K, of Professor Batho, can be proved to be $K = \frac{w\,t_2\,E}{p}\,k$, using the notation of the paper. Since k in Equations (26) and (27) is independent of the load, and K, according to Professor Batho, increases in proportion to the load on the joint, the two can agree only for one particular value of F. This value for some of the joints studied by Professor Batho is nearly equal to the working load; for others, it is considerably greater. Considering his expressions for k merely as first approximations the writer attaches no particular significance to these results, and mentions them only in view of questions raised by Professor Finlay and Mr. de Jonge. However, the fact that there is general agreement between the two investigations, conducted independently, and by different methods, is gratifying.

The writer quite agrees with Messrs Shearwood, Roop, and de Jonge when they state that the behavior of a riveted joint under variable load depends on its entire previous stress history; but he does not attempt to discuss here this phase of the question, limiting himself to a consideration of the action of a steady load and to the task of calling attention to the weaknesses of conventional design, the assumptions of which (using an apt expression by Professor Finlay), "if given sufficient time for their assimilation, tend to be confused with the eternal verities."

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Paper No. 1867

STUDY OF STILLING-BASIN DESIGN

By C. Maxwell Stanley,[1] Jun. Am. Soc. C. E.

With Discussion by Messrs. E. W. Lane, C. C. Inglis, F. Knapp, and C. Maxwell Stanley.

Synopsis

This paper presents the results of a study, made by means of hydraulic models, to rationalize the design of stilling basins, one of the several devices used for the prevention of scour at the toe of a dam. The relationships between the fundamental factors controlling the action of the stilling basin are carefully compared and, finally, a method of design based upon these relationships is presented.[2]

The stilling basin may be defined broadly as a pool of water held at the toe of a dam by a weir. Five distinct types of stilling-basin action and their characteristics are classified. The experimental data from the tests are studied to determine the relationships of basin length and depth, dam height, and quantity occurring for each of the five types of action. A more complete form of the paper has been placed in Engineering Societies Library, 33 West 39th Street, New York, N. Y., for reference.

Problem of Scour

The prevention of scour at the toe of an over-flow dam is a problem of importance to the designer, and upon its successful solution depends the safety of the dam. Although the solution of the problem is more urgent in dams constructed upon foundations of sand and gravel, which are easily eroded, it must not be overlooked in dams constructed upon seemingly solid

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Note.—Published in Novmbeer, 1932, *Proceedings*.

[1] Engr. (Young & Stanley, Inc.), Muscatine, Iowa.

[2] Prepared from a thesis presented to the University of Iowa in partial fulfillment of the requirements for the degree of Master of Science.

rock. Engineering literature contains too many reports of damage from erosion requiring costly repairs, if not resulting in disastrous failures of the dam itself.

A problem of such importance has naturally been studied to a considerable extent, but the theoretical foundation upon which to base an attack is obscure, and little has been done to formulate the basic laws. No general mathematical formula has been developed, which will allow even an approximate solution. As a result, solutions have been largely empirical, based on judgment and past experience. Their insufficiency has been indicated by the many cases of serious erosion, and although there have been notable examples of successful solution, the net result can scarcely be considered satisfactory.

Within recent years, many satisfactory solutions have resulted from model studies, which are to be highly commended and encouraged. The day is not far distant when experiments with models will be a necessary part of the design of every over-flow dam.

Even if a model study may be counted upon to solve a particular problem some rational method of design for protective devices would be extremely welcome. Such a method would be of great value in preliminary designs and estimates even if supplemented later by a model study.

THE STILLING BASIN

For the purposes of this paper, a stilling basin is defined as a pool of water retained below the toe of a dam by a weir, in order to dissipate energy contained in the water overflowing the spillway and thus to prevent serious erosion. The floor of the basin is assumed to be of concrete or other material that will prevent erosion at the toe where the sheet of water impinges. It should be pointed out, however, that natural conditions may create a pool of such dimensions that a weir is unnecessary, and the resulting actions will be similar to that described herein. A stilling basin thus defined may be deep or shallow, long or short. Naturally, the action will not be the same under these varying conditions of length and depth. The different actions fall into five types, each of which has certain characteristics differentiating it from the others, namely: I.—Roller Action; II.—Jump Action; III.—Incomplete Jump Action; IV.—Pool Action; and, V.—Spray Action.

Type I.—Roller Action.—As the swift stream of water coming down the face of the dam enters the pool of water in Fig. 1, it is deflected by the bucket and turned to a general horizontal direction. Friction on the floor and on the water above decreases the velocity and sets up an eddy or roller about a horizontal axis. This roller is full of air bubbles and slopes backward on top, slightly toward the incoming sheet of water. It absorbs and dissipates energy from the incoming sheet. The turbulence is out of the water before it reaches the weir, and the depth over the weir is just sufficient to discharge the incoming quantity. The only energy remaining is that of the static and velocity heads just above the weir, which is small compared to the incoming energy.

Type II.—Jump Action.—In the action of Type II (Fig. 2), the basin is longer than that of Type I, but in proportion to the quantity of water handled is not as deep. The general appearance is that of a hydraulic jump, as the usual indications of the jump are present. The basin is of sufficient length to allow the complete formation of this jump, with the result that all turbulence is out of the water before it reaches the weir, and it flows over smoothly. The remaining energy is that of the static and velocity heads just above the weir.

Type III.—Incomplete Jump Action.—This type of action is that occuring if the weir of Fig. 2 is moved toward the dam. The part toward the dam still gives every indication of being hydraulic-jump action, but the length is not sufficient to allow the complete formation of the jump.

The depth at the weir does not increase appreciably, but the water, instead of flowing smoothly over the weir, shoots in a more or less horizontal direction. The length clearly is too short for the proper formation of the jump. The remaining energy will be greater than that of Types I and II as the velocity over the weir is greater.

Type IV.—Pool Action.—Action of this type (Fig. 3) occurs if the weir of Fig. 1 is moved toward the dam. The basin is comparatively short and deep.

Instead of the smooth action of Type I the water shoots in and out of the basin and is thrown into the air. The pool surface is generally smooth except for a decided bump over the weir, and a slight roller is formed. The remaining energy is again greater than that of Types I and II as the water is thrown high into the air.

Type V.—Spray Action.—This type of action is illustrated in Fig. 4 and occurs if the basin is too shallow to hold the water. The length is of little importance as the sheet moves swiftly across the floor until it encounters the weir. It is then sprayed into the air. In this type, too, the remaining energy at the weir is greater than that of Types I and II.

These five types of stilling basin seem to classify the forms of action satisfactorily. The dividing lines between the types naturally are not clear cut and distinct, because one type merges into another. In Types I and II as contrasted to Types III, IV, and V, the basin is of such proportions that sufficient energy is dissipated to allow the water to flow smoothly over the weir. The energy remaining is definitely calculable, and its effects may easily be determined. Types I and II, therefore, are considered the more satisfactory kinds of action. It is the purpose of this study to define the conditions under which each of the types of action will occur and then to present a method of design by which a stilling basin may be proportioned to obtain any desired type of action.

While the ultimate test of the effectiveness of any stilling basin is its ability to prevent dangerous scouring, a clear knowledge of the factors that affect the operation of the basin and the relationships existing between these factors for satisfactory or unsatisfactory action will be helpful.

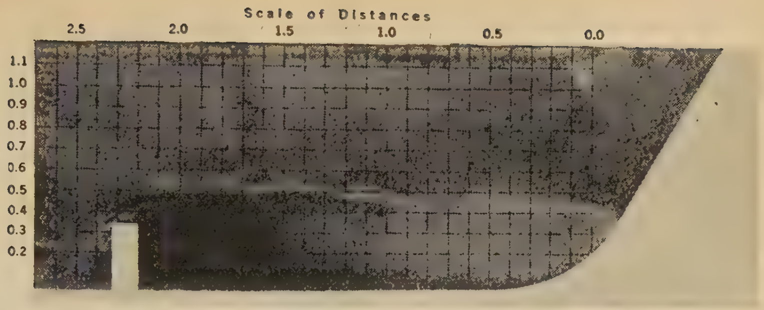
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FIG. 1.—TEST 125, TYPE I: STILLING BASIN, ROLLER ACTION.

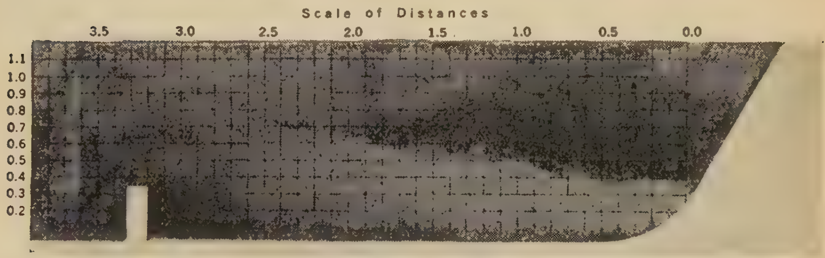
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FIG. 2.—TEST 476, TYPE II: STILLING BASIN, JUMP ACTION.

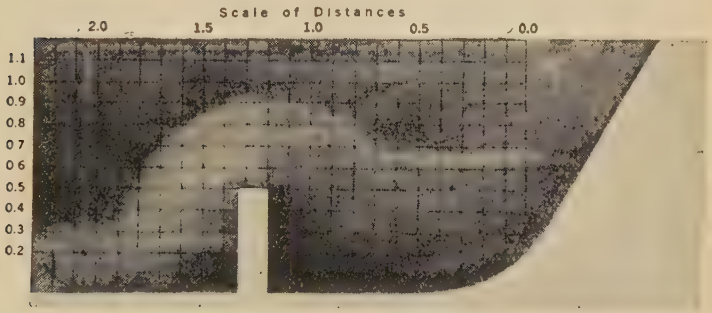
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FIG. 3.—TEST 462, TYPE IV: STILLING BASIN, POOL ACTION.

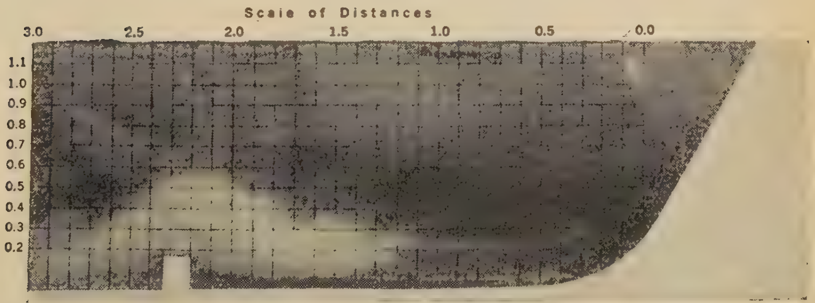
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FIG. 4.—TEST 453, TYPE V: STILLING BASIN, SPRAY ACTION.

TEST METHODS

The experiments were performed in a timber flume, 2 ft. wide and approximately 20 ft. long, with a glass-sided flume a little more than 6 ft. long at the lower end. (See Fig. 5.) Water was supplied by a pump with a capacity of 1 000 gal. per min., from a measuring basin under the floor of the laboratory to a tank (constant level) on the hill back of the laboratory. From this tank, the water flowed through a pipe line into the flume, over the model dam, and back into the measuring basin. Cross-connections were provided so that direct circulation could be used on the larger quantities. A Venturi meter was inserted in the pipe line and a valve was placed at the inlet to the flume. Four sets of baffles, set about 13 ft. up stream from the model dam, successfully smoothed the flow. A hook-gauge well was located 6 ft. up stream from the model.

The dam was built of wood with a cross-section as shown for Series *B* (Fig. 5). Its surface was smoothed and painted carefully with several coats of shellac. Alterations were made at the toe to obtain the cross-sections of Series *A* and *C* (Fig. 5). The weirs that formed the stilling basins were constructed of 2-in. material and were fastened to the floor of the glass flume. Their rating curves, determined experimentally, are shown in Fig. 6, in which, D_w denotes depth of water at the weir (height of weir plus head on the weir) required for smooth flow over the weir, and D_j is the depth of water required in the basin for the formation of a hydraulic jump.

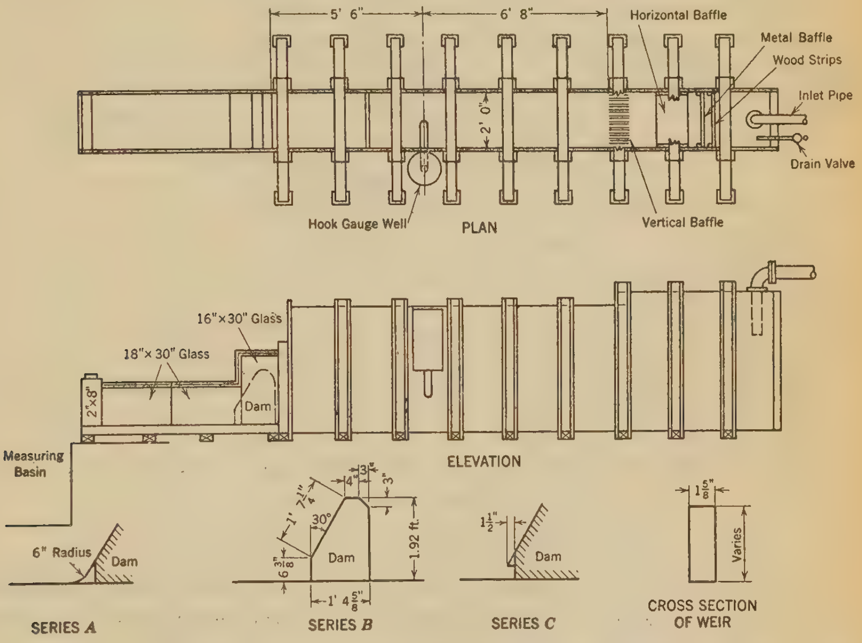

5' 6"
6' 8"
Horizontal Baffle
Metal Baffle
Wood Strips
Inlet Pipe
2' 0"
Drain Valve
Hook Gauge Well
PLAN
Vertical Baffle
16"×30" Glass
18"×30" Glass
2"×8"
Dam
Measuring Basin
ELEVATION
6" Radius
Dam
SERIES A
3"
4"
3"
$7\frac{1}{4}$"
1'
30°
Dam
1.92 ft.
$6\frac{3}{8}$"
1' $4\frac{5}{8}$"
SERIES B
$1\frac{1}{2}$"
Dam
SERIES C
$1\frac{5}{8}$"
Varies
CROSS SECTION OF WEIR

FIG. 5.—DETAILS OF TIMBER FLUME AND MODELS.

In running the tests a weir of given height was fastened to the floor at a given distance from the dam. The water was then circulated through the system and the quantity over the dam was adjusted by the intake valve. After conditions had become steady, readings were taken of the quantity and depth, and notes were made of the stilling-basin action. During the first runs, the Venturi meter was used to determine the quantity of water flowing over the dam and simultaneous readings were taken of the hook-gauge. As soon as a satisfactory rating curve had been built up for the dam, the Venturi meter readings were discontinued, except as an occasional check.

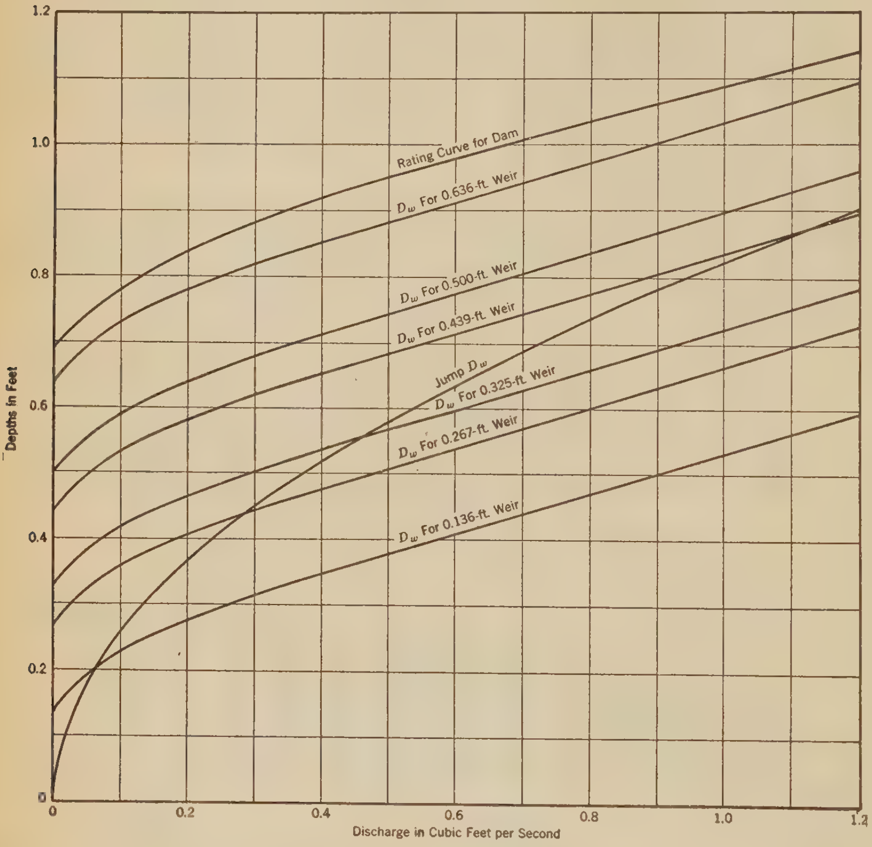


FIG. 6.—RATING CURVES FOR DAM AND WEIRS.

All readings of depth and length were taken by direct observation of the water surface through the glass by a grid painted on the glass with horizontal and vertical lines spaced 0.1 ft. apart. Notes were recorded regarding the action of the stilling basin and from these data the action was classified into the five types previously described. On many tests, sketches were made of the action in the stilling basin, and, in some cases, photographs were taken.

The tests were divided into three series (*A, B,* and *C*) for three variations in the form at the toe of the dam. (See Fig. 5.) The method of testing was the same in each series. The data include the height of weir, test number, length, quantity, and type of action from the observed notes. The computed values of D_j, D_w, and the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, are also shown, in which, L is the length of the basin. Table 1 illustrates the manner in which data for 350 tests were assembled and arranged for use in this study.

Rating curves were built up experimentally for weirs of different heights and from these the values of D_w were obtained. The depth of water required for the formation of a hydraulic jump for various discharges was computed (assuming no friction on the down-stream face of the dam) by the hydraulic-jump formula:

$$D_j = -\frac{D_1}{2} \pm \sqrt{\frac{2\,V_1^2\,D_1}{g} + \frac{D_1^2}{4}} \quad \dots\dots\dots\dots\dots\dots(1)$$

in which, D_1 and V_1 represent the theoretical depth and velocities at the toe of the dam, and g is the acceleration due to gravity. The values of D_j were then plotted in Fig. 6. The ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, were computed from the complete record of which Table 1 is a condensed example.

METHOD OF COMPARING TESTS

A number of methods of comparing the test data have been tried in an attempt to find one that would segregate the tests according to the observed types of action. The most satisfactory method proved to be the plotting of two ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, for each test. These ratios have a definite meaning which must be kept clearly in mind if the test results are to be interpreted properly.

The first ratio, $\frac{D_w}{D_j}$, is the depth required for smooth flow over the weir with a given discharge, divided by the depth required in the basin for the formation of a hydraulic jump with the same discharge. Let W be the height of the weir and h_w, the required head on the weir; then, D_w will be equal to $W + h_w$. Since D_j is determined from the hydraulic-jump formulas, it depends on the velocity at the toe of the dam and on the quantity of water flowing over the dam. The velocity is dependent upon the height, slope, friction, etc., of the dam. A ratio of unity means that the basin is proportioned so that the depth required for the given discharge over the weir is just that required for the formation of a hydraulic jump under the existing conditions. Ratios greater or less than unity mean that the basin is deeper or shallower than required for the formation of a jump.

TABLE 1.—TYPICAL ARRANGEMENT OF OBSERVED AND COMPUTED STILLING-BASIN DATA, SERIES *A*

Test No. (1)	OBSERVED Length, in feet. (2)	 Discharge, in cubic feet per second (3)	 Depth, D, in feet. (4)	COMPUTED Depth for hydraulic jump, D_j, in feet (5)	 Depth for weir, D_w, in feet (6)	 Ratio, $\frac{D_w}{D_j}$ (7)	 Ratio, $\frac{L}{D_w}$ (8)	 Action (9)
WEIR, 0.136 FOOT HIGH								
322......	1.2	0.169	0.26	0.34	0.27	0.79	4.45	II
323......	1.2	0.243	0.34	0.41	0.30	0.73	4.00	V
325......	1.2	0.667	0.85	0.68	0.43	0.63	2.79	V
326......	1.7	0.160	....	0.33	0.26	0.79	6.55	II
328......	1.7	0.197	....	0.37	0.28	0.76	6.07	V
329......	1.7	0.334	....	0.48	0.33	0.69	5.15	V
330......	2.2	0.176	0.28	0.35	0.27	0.77	8.15	II
331......	2.2	0.238	0.30	0.40	0.30	0.75	6.60	V
453......	2.2	0.345	....	0.48	0.33	0.69	6.65	V
332......	3.7	0.165	0.37	0.34	0.27	0.79	13.7	II
333......	3.7	0.195	0.30	0.37	0.28	0.76	13.2	II
336......	3.7	0.225	....	0.39	0.29	0.74	12.7	V
WEIR, 0.267 FOOT HIGH								
272......	0.7	0.042	0.33	0.15	0.31	2.06	2.26	I
273......	0.7	0.079	0.39	0.22	0.34	1.54	2.06	I
274....	0.7	0.260	0.63	0.42	0.43	1.02	1.63	IV
267......	1.2	0.078	0.36	0.22	0.34	1.54	3.53	I
268......	1.2	0.127	0.38	0.30	0.38	1.26	3.16	I
269......	1.2	0.173	0.42	0.35	0.40	1.14	3.00	IV
271......	1.2	0.522	0.75	0.60	0.52	0.87	2.31	IV
262......	1.7	0.117	0.38	0.28	0.37	1.32	4.60	I
263......	1.7	0.206	0.43	0.38	0.41	1.08	4.15	I
264......	1.7	0.308	0.47	0.46	0.45	0.98	3.78	IV
266......	1.7	0.652	0.72	0.67	0.56	0.84	3.04	IV
255......	2.2	0.153	0.39	0.33	0.39	1.18	5.64	I
256......	2.2	0.208	0.42	0.38	0.41	1.08	5.36	II
258......	2.2	0.413	0.52	0.53	0.48	0.91	4.58	II
261......	2.2	0.847	0.72	0.76	0.62	0.82	3.55	III
225......	2.7	0.103	0.36	0.26	0.36	1.38	7.50	I
226......	2.7	0.169	0.40	0.34	0.40	1.18	6.75	I
226......	2.7	0.302	0.44	0.45	0.45	1.00	6.00	II
229......	2.7	0.461	0.50	0.56	0.50	0.89	5.40	II
230......	2.7	0.543	0.53	0.61	0.52	0.85	5.18	III
235......	2.7	1.220	0.85	0.90	0.74	0.82	3.65	III
236......	3.2	0.290	0.44	0.44	0.44	1.00	7.27	II
239......	3.2	0.503	0.52	0.59	0.51	0.86	6.27	II
240......	3.2	0.585	0.55	0.64	0.54	0.84	5.93	III
242......	3.2	0.958	0.78	0.81	0.65	0.80	4.93	III
243......	3.7	0.059	0.33	0.18	0.33	1.64	11.2	I
244......	3.7	0.181	0.41	0.35	0.40	1.14	9.3	II
247......	3.7	0.538	0.53	0.61	0.53	0.87	7.00	II
248......	3.7	0.684	0.56	0.69	0.57	0.83	6.50	V
249......	3.7	0.894	0.58	0.78	0.63	0.81	5.88	V
250......	4.2	0.248	0.43	0.41	0.43	1.05	9.78	II
252......	4.2	0.496	0.52	0.58	0.51	0.88	8.25	II
253......	4.2	0.591	0.52	0.64	0.54	0.84	7.79	V
254......	4.2	0.772	0.53	0.73	0.59	0.81	7.13	V
WEIR, 0.325 FOOT HIGH								
143......	0.7	0.056	0.40	0.18	0.38	2.11	1.84	I
142......	0.7	0.118	0.47	0.28	0.42	1.50	1.67	IV
145......	0.7	0.785	1.27	0.74	0.66	0.89	1.06	IV
137......	1.2	0.111	0.44	0.27	0.42	1.55	2.86	I
138......	1.2	0.156	0.50	0.33	0.45	1.36	2.67	IV
140......	1.2	0.785	1.15	0.74	0.65	0.88	1.85	IV

The other ratio, $\frac{L}{D_w}$, is the length of the basin divided by the depth required at the weir and is an index of the remainder of the proportions of the basin. When $\frac{D_w}{D_j}$ is unity, the second ratio, $\frac{L}{D_w}$, becomes $\frac{L}{D_j}$, or the length divided by the depth required for the formation of the jump.

These ratios thus tie the action of the stilling basin closely to the fundamental factors of the hydraulic jump.

Test Results, Series *A*

Each test of Series *A* has been plotted in Fig. 7, according to its value of the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, with the resulting type of action designated by the mark used in plotting the data. It is easy to draw dividing lines between the five types of action. This is proof that there are definite types of stilling-basin action and that these may be segregated according to the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$.

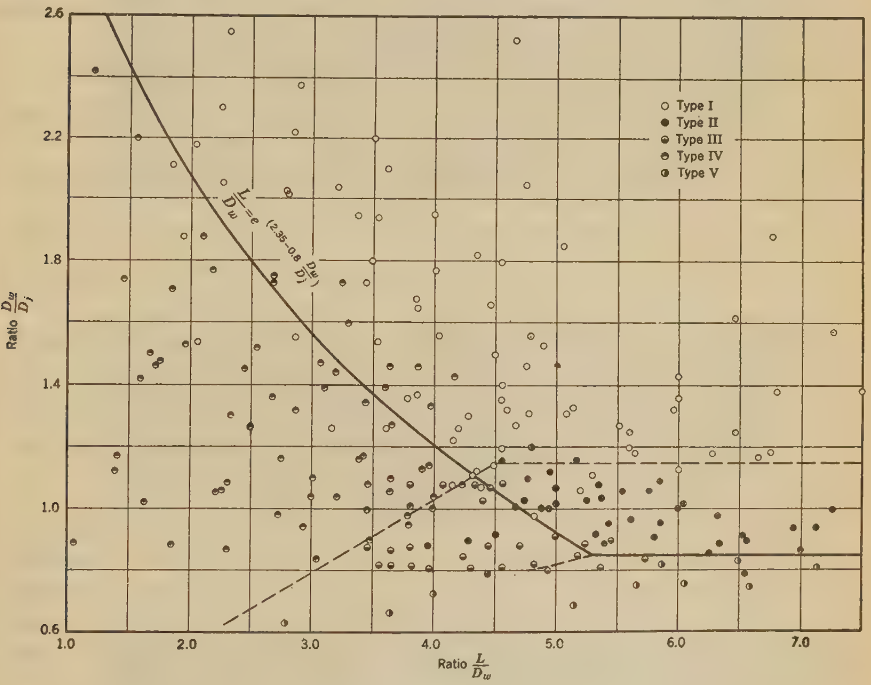


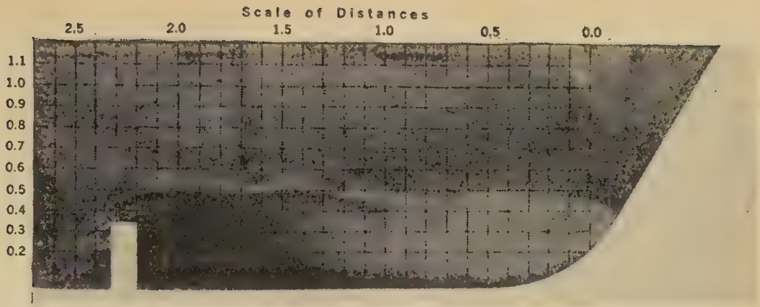
Fig. 7.—Ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, Series *A*.

The solid curve in Fig. 7 marks the division between Types I and II which are above the line, and Types III, IV, and V, which are below it. Types I and II may be considered satisfactory in that stilling-basin dimensions have such proportions that the excess energy is absorbed completely from the water, allowing it to flow smoothly over the weir. In Types III, IV, and V, these requirements are not fulfilled, and the basin does not function satisfactorily.

An analysis of Fig. 7 shows that Type I may be expected to occur whenever the dimensions of the bay are such that the point plots above the curve and above the line, $\frac{D_w}{D_j} = 1.15$. The action classified as Type II occurs when the ratio, $\frac{D_w}{D_j}$, is between 0.85 and 1.15, or substantially equal to unity, and the ratio, $\frac{L}{D_w}$, exceeds about 4.5. In other words, the depth of the pool must be that required for the formation of a hydraulic jump and the length of the pool must be not less than about 4½ times the depth of the jump. These values check with the commonly accepted requirements of a hydraulic jump, thus confirming the idea that Type II is essentially a hydraulic jump.

Types I and II, which are considered satisfactory, occur whenever the ratios plot above the curve until it reaches a value of $\frac{L}{D_w} = 5.31$. After that, the curve is a straight line, $\frac{D_w}{D_j} = 0.85$. Whenever the ratios plot below this curve, action will be unsatisfactory and will be of the classes characterized as Types III, IV, or V. Type III occurs when the ratio, $\frac{D_w}{D_j}$, is between 0.85 and 1.15, and $\frac{L}{D_w}$ is slightly less than that required for a jump. Type IV action occurs whenever the length or the depth is below that required for Type I. Type V action occurs whenever the ratio, $\frac{D_w}{D_j}$, is less than about 0.85 and occurs regardless of the length of the pool.

Figs. 8, 9, and 10 have been prepared to show the effect upon the basin action of changes in the fundamental variables. It is to be noted particularly that the tests are grouped in sets and that Figs. 9(*a*), 9(*b*), and 9(*c*) are parts of two sets; thus, Set 1 comprises Figs. 8(*a*), 9(*c*) and 8(*b*); Set 2 is represented by Fig. 9(*b*), as an example; Set 3 is represented by Figs. 8(*c*) and 8(*d*); Set 4 is represented by Figs 9(*a*) and 10(*b*); Set 5 includes Figs. 9(*b*), 9(*c*), and 9(*d*); Set 6 comprises Figs. 10(*c*) and 10(*a*); Set 7 is represented by Figs. 9(*c*), 10(*a*), and 10(*b*); and Set 8 is shown in Figs. 10(*c*), 9(*d*), and 10(*d*). In each set only one variable has been changed and the others have been held constant. In Fig. 11, the ratios applying to the illustrated tests are plotted and are studied in relation to the dividing line between satisfactory and unsatisfactory action in Series *A*. A study of the views and curves clearly defines the effect of changes in length, depth, height of weir, and discharge.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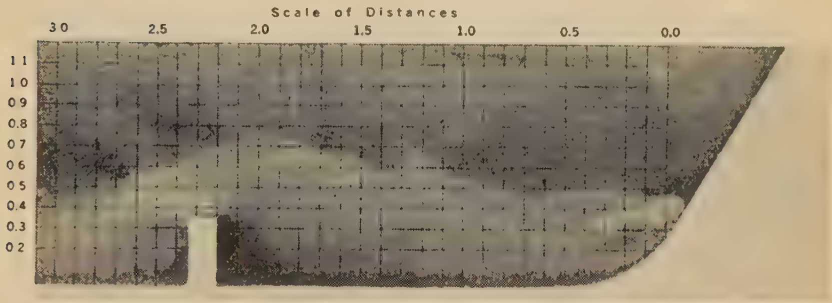
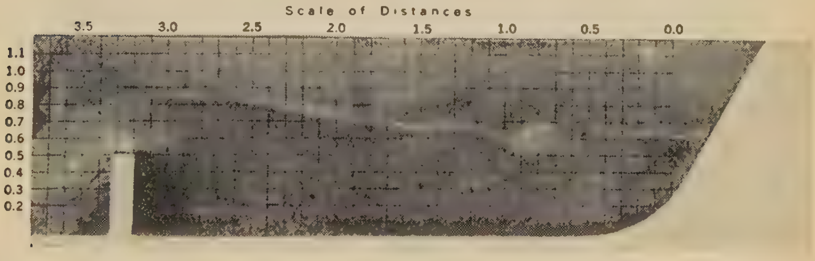
0.5
0.4
0.3
0.2

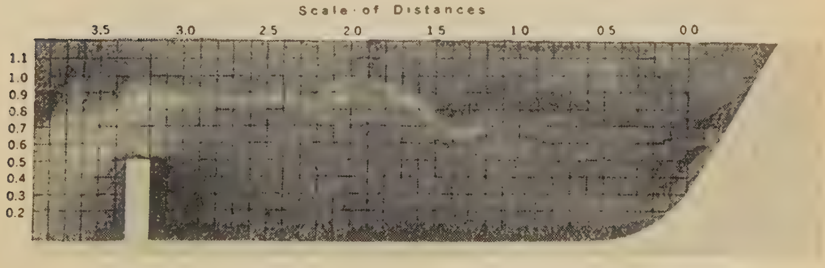
(*a*) SET 1, TEST 125, TYPE I: $L = 2.2$; $W = 0.325$; $Q = 0.193$.


Scale of Distances
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*b*) SET 1, TEST 129, TYPE IV: $L = 2.2$; $W = 0.325$; $Q = 0.720$.

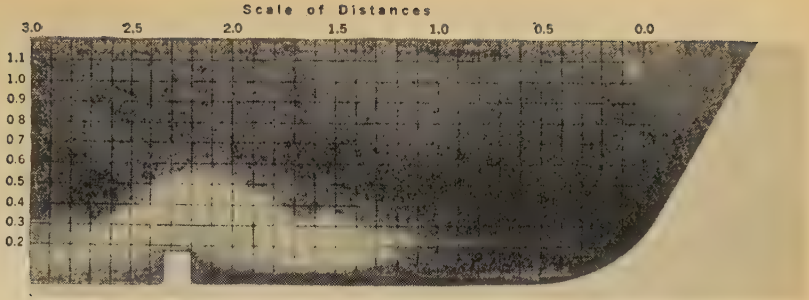

Scale of Distances
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*c*) SET 3, TEST 477, TYPE I: $L = 3.2$; $W = 0.500$; $Q = 0.610$.

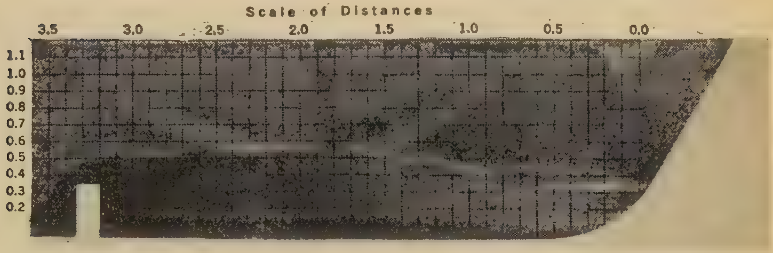

Scale of Distances
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*d*) SET 3, TEST 478, TYPE IV: $L = 3.2$; $W = 0.500$; $Q = 0.868$.

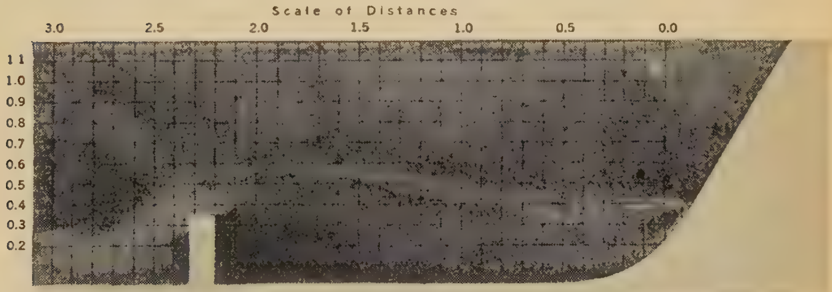
FIG. 8.—EFFECT OF INCREASING THE DISCHARGE, Q.


Scale of Distances
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

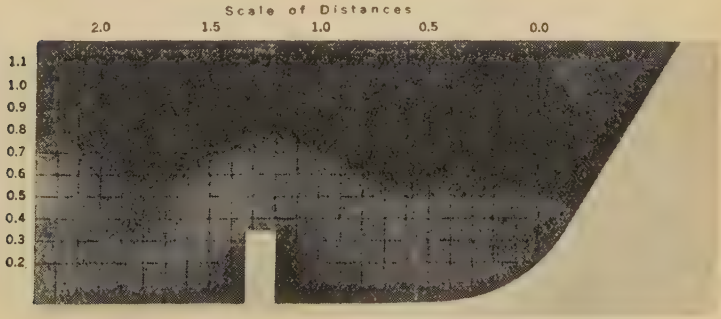
(*a*) Set 4, Type V: $L = 2.2$; $W = 0.136$; $Q = 0.346$; also, Set 7.


Scale of Distances
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*b*) Set 5, Type II: $L = 3.2$; $W = 0.325$; $Q = 0.346$; also, Set 2.

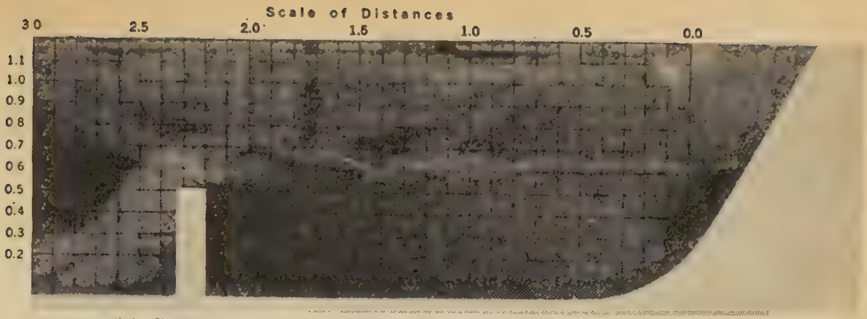

Scale of Distances
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*c*) Set 5, Type IV: $L = 2.2$; $W = 0.325$; $Q = 0.346$; also, Sets 1 and 7.

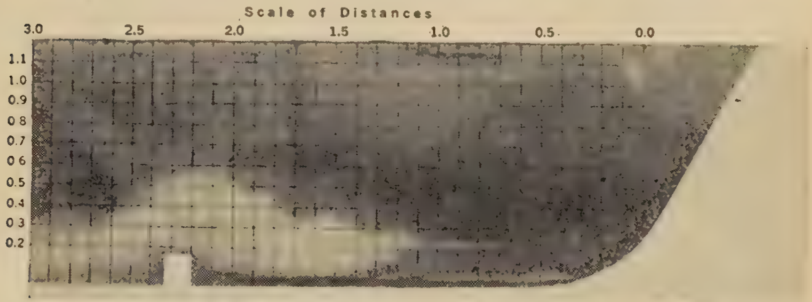

Scale of Distances
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*d*) Set 5, Type IV: $L = 1.2$; $W = 0.325$; $Q = 0.346$; also, Set 8.

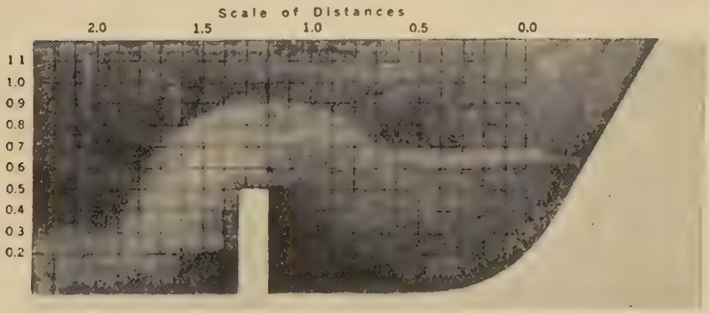
Fig. 9.—Effect of Increasing the Length, *L*, of the Basin.


Scale of Distances
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

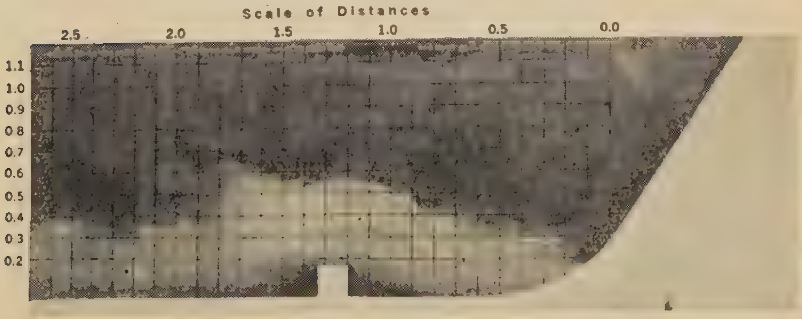
(*a*) SET 7, TYPE IV: $L = 2.2$; $W = 0.500$; $Q = 0.346$; ALSO, SET 6.


Scale of Distances
3.0
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*b*) SET 7, TYPE V: $L = 2.2$; $W = 0.136$; $Q = 0.346$; ALSO, SET 4.


Scale of Distances
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*c*) SET 8, TYPE IV: $L = 1.8$; $W = 0.500$; $Q = 0.346$; ALSO, SET 6.


Scale of Distances
2.5
2.0
1.5
1.0
0.5
0.0
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2

(*d*) SET 8, TYPE V: $L = 1.2$; $W = 0.136$; $Q = 0.346$.

FIG. 10.—EFFECT OF DECREASING HEIGHT OF WEIR, W.

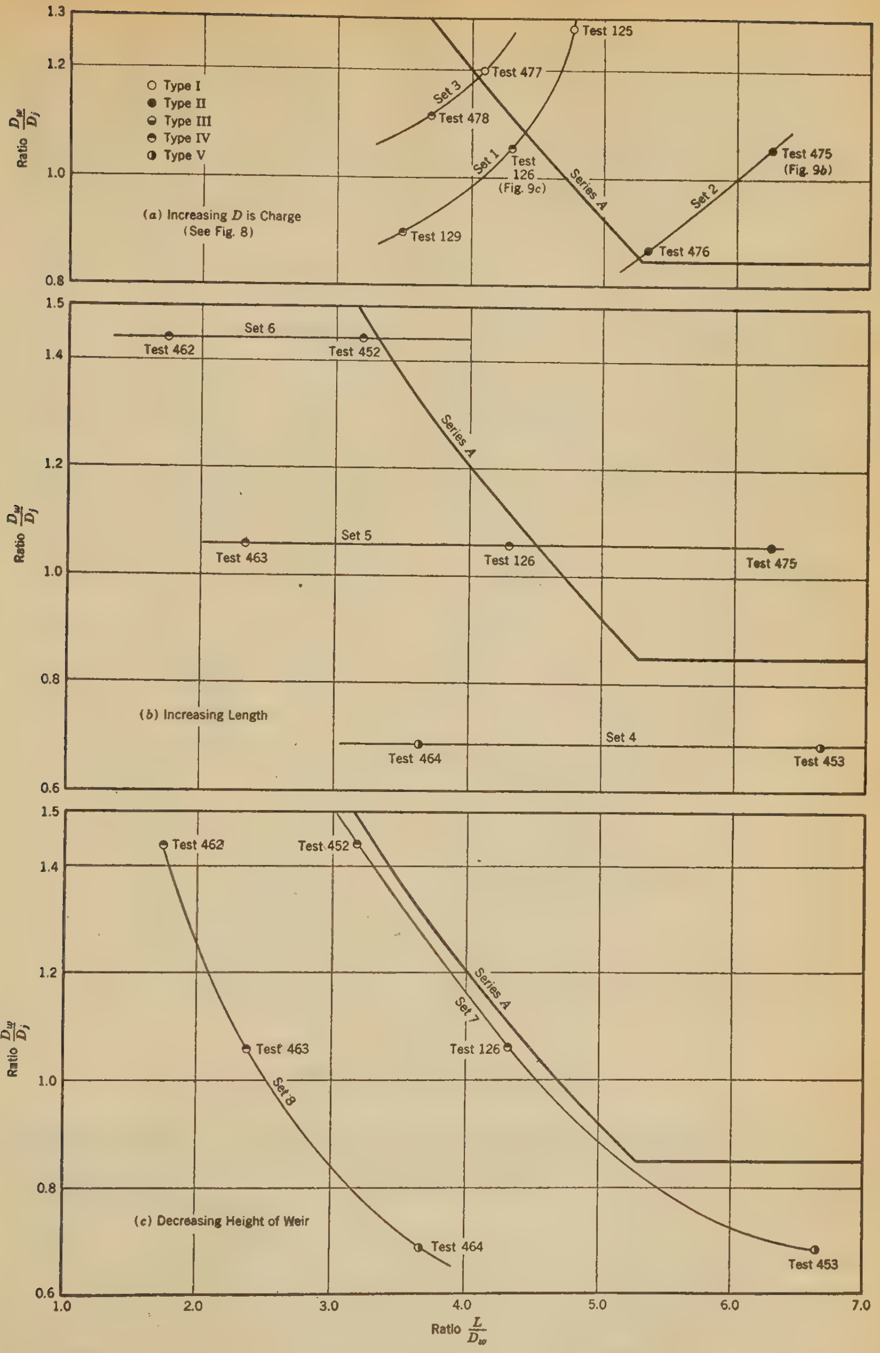


FIG. 11.—RELATION BETWEEN RATIOS, $\frac{L}{Dw}$ AND $\frac{Dw}{Dj}$, SERIES A; (a) INCREASING DISCHARGE (SEE FIG. 8); (b) INCREASING LENGTH (SEE FIG. 9); AND (c) DECREASING HEIGHT OF WEIR (SEE FIG. 10).

TEST RESULTS, SERIES *B*

The tests of Series *B* were performed similar to those of Series *A*, except that the 6-in. radius at the toe of the dam was removed, leaving the dam in its original cross-section as shown on Fig. 5. No attempt was made to run as complete a set of tests with Series *B*, as it was desired merely to check the curve of Series *A*. Nevertheless, the tests were run covering the entire range used in Series *A*. Fig. 12 shows the tests of Series *B* plotted with ratios of $\frac{D_w}{D_j}$ against ratios of $\frac{L}{D_w}$. The solid curve seems to satisfy the data down to the point where $\frac{D_w}{D_j} = 0.8$. This curve is identical with the curve of Series *A*, except that a constant, 1.10, has been subtracted from the exponential function indicating that the basin of Series *B* is considerably more effective than that of Series *A*.

It was considered unnecessary to draw the dividing line between Types I and II and between Types III, IV, and V. It seems logical that the basin of Series *B* should be more effective that that of Series *A*, although the magnitude of the difference was rather surprising. With the 6-in. radius of Series *A*, the sheet of incoming water is turned to a horizontal direction with little disturbance, while with the vertical cut-off it impinges with considerable force upon the floor of the bay. A roller or eddy is also created between the vertical face of the dam and the sheet of incoming water.

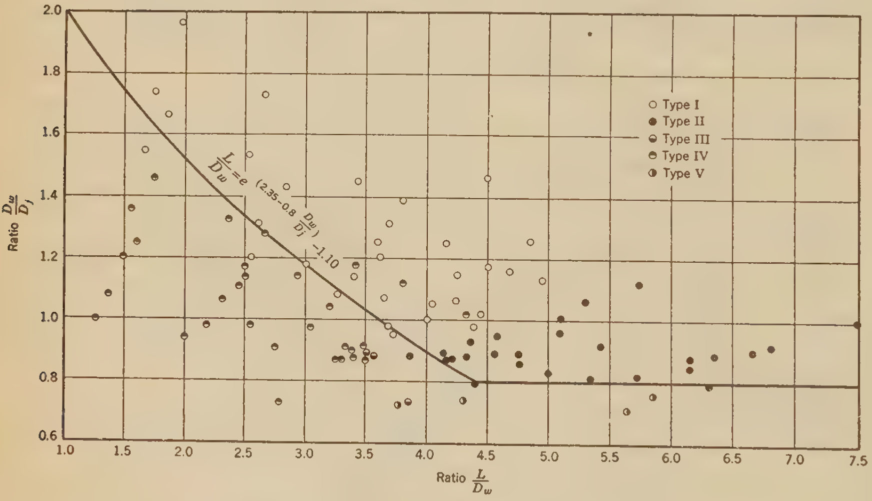


FIG. 12.—RATIOS, $\frac{D_w}{D_j}$ AND $\frac{L}{D_w}$, SERIES *B*.

One other reason for the difference in the effectiveness of the two bays is the method of measuring the length of the bay. In both cases, the measurement is made from the up-stream face of the weir to the intersection of the down-stream slope of the dam with the floor of the flume. This means that with the vertical cut-off, the basin is actually about 0.3 ft. longer than the basin of Series *A* for a given position of the weir. This variation in length accounts for part of the difference in operation, but is not sufficient to account for it entirely. It is obvious that the bay of Series *B* with the vertical cut-off is more effective than that of Series *A*. No attempt has been made to show curves and views of Series *B* such as those shown of Series *A* in Figs. 8, 9, and 10, because they would be similar.

TEST RESULTS, SERIES *C*

The tests of Series *C* and the curve for Series *B* are plotted in Fig. 13 by ratios of $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$. The basin of Series *C* was similar to that of Series *B*, except that a triangular strip of moulding was placed on the vertical face, as shown in Fig 5, to approximate a design of the tumble bay proposed[3] by A. F. Meyer, M. Am. Soc. C. E. Only a few tests were run in Series *C*; but there is no indication that the curve for this series should be other than that of Series *B*, indicating that the Meyer bay has its effectiveness in the vertical cut-off rather than in the overhanging lip.

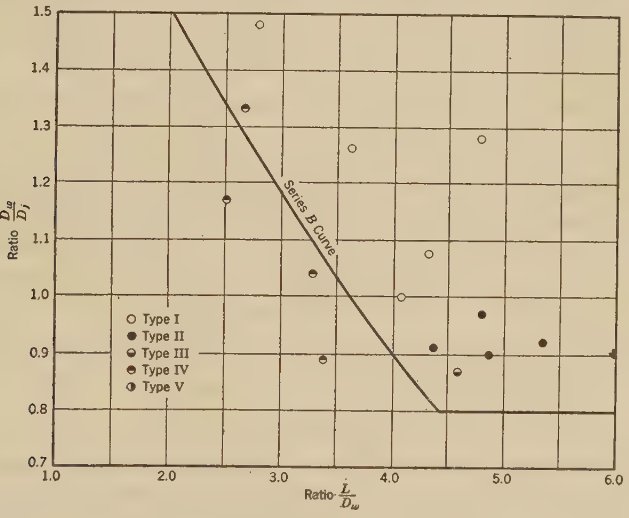


FIG. 13.—RATIOS, $\frac{D_w}{D_j}$ AND $\frac{L}{D_w}$, SERIES *C*.

EFFECT OF SHAPE OF DAM

The difference between the curves of Series *A* and Series *B* indicates that the shape of the dam has considerable effect upon the action of the stilling basin. The three forms tested in Series *A*, *B*, and *C* (Fig. 5) are entirely too few to set up any definite relationship as to the effect of the shape of the dam. The model dam was built with a down-stream slope of 60° with the horizontal, and there was no opportunity to try other slopes. Differences in action would be expected with differences in slope of the down-stream face of the dam as well as in differences in construction at the toe of the dam. It is to be expected that the steeper the slope, the more effective will be the action of the basin, at least for action of Type I. In the case of a hydraulic-jump action of Type II, it is possible that a flatter slope would allow the jump to form more easily, thus decreasing the length of basin needed for the complete formation of the jump. Certainly, it may be said that the vertical cut-off principle, as illustrated in Series *B*, is more effective than the bucket of Series *A*. There is need of further research to determine the effect of the shape of the dam on stilling-basin action.

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[3] "High Velocity Discharge of Overfall Dams and Forms of Spillway Profile," *Engineering News-Record*, April 9, 1925.

EFFECT OF SHAPE OF WEIR

All the tests were run with weirs of similar cross-section, that is, one with a vertical face, horizontal top, and a square corner on the up-stream face. There seems to be no reason to believe, however, that change in the shape of the weir would alter the basic relationship of type of action and the ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$. A change in the shape of the weir would change only the value of D_w. It may be assumed that the weirs used in the test had a coefficient of about 3.3, and that a weir with properly designed crest might have a coefficient of 3.7. This would result in a slight decrease of the value of h_w for a given quantity, but as D_w is $W + h_w$ for a given quantity, it would change D_w even less. The result would be an effective height of weir less than that of the test. As the difference in heights of weir produced no differences in the fundamental relations, the differences in shape of weir could not affect the results.

EFFECT OF SUBMERGENCE OF WEIR

A few experiments were made in an attempt to establish quantitatively the effect of submerging the weir. The number of tests was too few to arrive at any definite conclusions. However, it may be said that the immediate effect is to require a greater head on the weir to discharge a given quantity of water. As a result the value of D_w is increased and the ratio, $\frac{D_w}{D_j}$, will also increase with a constant quantity. The other ratio, $\frac{L}{D_w}$, will be decreased slightly but, as D_j is normally much smaller than L, the change in the ratio, $\frac{D_w}{D_j}$, will be of greater magnitude than that of the ratio, $\frac{L}{D_w}$. As a result the depth of the basin is sufficient to handle a greater quantity; hence, the capacity to dissipate energy is increased by submergence of the weir.

INTERPRETATION OF TEST RESULTS

The conclusions presented herein may be summarized, as follows:

(1) The action of the stilling basin is closely connected to the phenomenon of the hydraulic jump.

(2) The ratios, $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, serve satisfactorily to classify the action of a stilling basin into the five types described.

(3) For satisfactory action of a stilling basin, the ratio, $\frac{D_w}{D_j}$, must be greater than 0.85.

(4) For any value of $\frac{D_w}{D_j}$ greater than 0.85, there is a corresponding minimum value of the ratio, $\frac{L}{D_w}$, that determines the minimum proportions of a stilling basin for satisfactory action. As the ratio, $\frac{D_w}{D_j}$, increases the ratio, $\frac{L}{D_w}$, decreases.

(5) Increasing the length of a basin to obtain a ratio of $\frac{L}{D_w}$ greater than this minimum value, does not improve the action. The ratio, $\frac{L}{D_w}$, need never exceed 4.0 or 5.0, depending upon the ratio of $\frac{D_w}{D_j}$, and upon the design of the dam.

DESIGN OF STILLING BASINS

The design of a stilling basin for any given dam is naturally dependent upon local conditions. A number of basins with varied proportions of depth and length $\left(\text{that is, varying ratios of } \frac{D_w}{D_j} \text{ and } \frac{L}{D_w}\right)$ may be designed to meet a particular height of dam and discharge, as has been discussed herein. The selection of the proportions for a given dam will depend principally upon foundation conditions. Whether or not the basin should be relatively deep or shallow is principally an economic question, the solution of which will depend on the elevation at which satisfactory foundations may be obtained for the dam and the stilling basin. The problem of whether or not the weir is to be submerged will also be principally one of economics; so, also, will be the question of the type of construction to be used at the toe of the dam. Although the vertical cut-off requires a shorter basin, it is accompanied by an additional quantity of concrete in the dam itself. Hence, no general design may be presented to meet all conditions, and satisfactory action will be obtained if the recommended proportions of the basin are used; but the actual design will differ with local conditions.

For design purposes, the solid curves of Figs. 7 and 12 may be plotted on one sheet as a guide to show the minimum ratios of $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, for action with dams constructed as in Series *A* and *B*. The required values of D_j for various heights of dam and various depths of discharge over the dam may be taken from a set of curves computed by Equation (1). The writer suggests plotting height of dam against D_j as ordinates for various values of head on the crest. The height of dam is taken as the vertical distance from the floor of the basin to the crest of the dam. The proportions for basins in which the action will be satisfactory may be readily determined from such curves.

Need of Further Research

In conclusion, the writer wishes to urge further research on this problem. The effect of submergence upon the capacity of a given basin to dissipate energy should be studied further. In studying the effects of the shape of a dam, various slopes as well as various constructions at the toe should be investigated. A series of experiments to determine the resulting scour below the weir for various basins, proportioned from the data included in this paper, would be desirable. Experimentation to determine what part of the basin requires a concrete floor to prevent erosion within the basin should also be made. Such research will lead to additional data of value in establishing a rational method of designing protective devices for overflow dams.

DISCUSSION

E. W. Lane,[4] M. Am. Soc. C. E. (by letter).—The best form of protection against scour at the toe of an overflow dam depends on the relative elevation, over the full range of discharge anticipated, of the natural tail-water level below the dam and the water-surface elevation required to form the hydraulic jump on a horizontal apron at the stream-bed level. When the tail-water level is below that required to form the jump, the best form of protection often is to increase the depth of the tail-water, either by increasing the water depth by excavation, or by raising the tail-water level by a secondary dam. For the latter case, if the flow over the secondary weir is not effected by the tail-water level down stream, the data contained in Mr. Stanley's paper are of great value to the designing engineer, and the profession is indebted to him for making it available. Since the tail-water level can rise above the crest of the secondary dam—if ogee in shape, two-thirds as high as the head-water—without materially influencing the discharge, the data will apply directly to a large range of conditions.

An extensive series of model tests similar to those of Mr. Stanley was made in 1930-31 for the design of the spillway of the Cle Elum Dam of the U. S. Bureau of Reclamation. This spillway was designed for a maximum discharge of 40 000 sec-ft and a water surface drop of 110 ft. It was of the trough type, controlled at the upper end by gates. The width at the upper end was 200 ft. At the middle this was narrowed to 100 ft and expanded to 200 ft at the lower end, where it extended the stilling pool. The trough was 900 ft long, and its shape as well as that of the stilling pool was developed by model tests. As there was no rock within reasonable depth, the entire structure was founded on gravel. The height of the tail-water was insufficient to form the hydraulic jump on an apron at stream-bed level, but instead of building a secondary weir to secure the depth necessary for jump formation it was obtained by depressing the pool bottom 11 ft below the river-channel bottom.

An extensive series of tests was undertaken to determine the best form of stilling basin. A model of the spillway on a 1:50 scale was set up in the hydraulic laboratory of the Colorado Agricultural College, at Fort Collins, Colo. The effectiveness of the various forms of stilling pool was determined by the scouring effects in a bin in which sand was formed to simulate the river bed down stream from the pool. Preliminary tests showed that duplicate runs would produce the same scour results, that the amount of compacting of the sand was of little importance, and that a run of 1 hr was sufficient to secure practically the same result as a flood of great duration in the spillway.

The first design of the stilling pool provided a horizontal floor, 102 ft long, down stream from which was a section 44 ft long sloping upward

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[4] Research Engr., U. S. Bureau of Reclamation, Denver, Colo.

11 ft to river-bottom level, followed by a 10-ft level floor. The tests showed considerable scour at the end of this floor, which could not be corrected by sills. A steeper upward sloping section produced even less desirable results. Much better success was obtained by making the floor level throughout, 11 ft below river-bed level, and permitting the rise from the pool floor to the river-bed level to be formed in the river bed down stream from the pool.

Although fairly satisfactory results were secured with a simple pool with a level floor, the scour was less if some form of sill was placed at the down-stream end of the floor. These sills were in no sense secondary weirs, as were those used in the author's experiments, since they did not control, or even materially influence, the tail-water level. The height of most of them was only about 30% of the water depth. Their function was to form an eddy behind them, at the down-stream edge of the floor in which the current in contact with the river bed moved up stream and dragged bed material up toward the end of the floor, rather than scouring it away.

A number of types of sill were used. The one on which most of the tests were made was the dentated type introduced by Dr. Theodor Rehbock.[5] Another had an ordinary ogee dam section, and a third was trapezoidal in section with a 45° slope on its up-stream side and with a vertical down-stream face. All these sills were located at the down-stream edge of the apron. Most of them had a height of 10 ft. Tests were made, however, which showed that greater scour occurred with smaller heights and less with greater heights. The 10-ft height was used because the volumes of higher weirs, particularly in the case of the dentated sill, became excessive for great heights. All these forms gave good results, those obtained with the dentated sill being slightly better than the others, and the dentated form was adopted for the structure.

Tests were made to determine the best slope for the floor entering the stilling pool. Slopes of 1 on 1.5, 1 on 2, 1 on 3, and 1 on 4, were tested. In most cases the scour increased as the slope became steeper; it is believed that this was due to the fact that the hydraulic jump forms more perfectly with the flatter slopes. With the dentated sill the scour was practically the same at all slopes and since other considerations favored steeper slopes, the 1 on 1.5 slope was adopted for the structure. Tests also showed that with a curved surface between the sloping incoming floor and the level pool bottom less scour was produced than when there was no such transition. This also is believed to be due to the fact that the curved surface tended toward the formation of a more perfect jump.

In order to secure the greatest economy in the construction of the pool, extensive experiments were performed to determine the best combination of width, depth, and length. Experiments were made with pool widths corresponding to 200, 150, and 120 ft. For each width three or four pool depths were used and with each depth, several lengths were tested. In all, thirty-eight conditions were investigated. The 10-ft dentated sill was used in all tests and the discharge and tail-water level was that corresponding to the maximum expected flood. The scour of the bed down stream from the floor was slight in all cases, but the action on the banks differed considerably. The results were unusually consistent, which gave considerable confidence in them.

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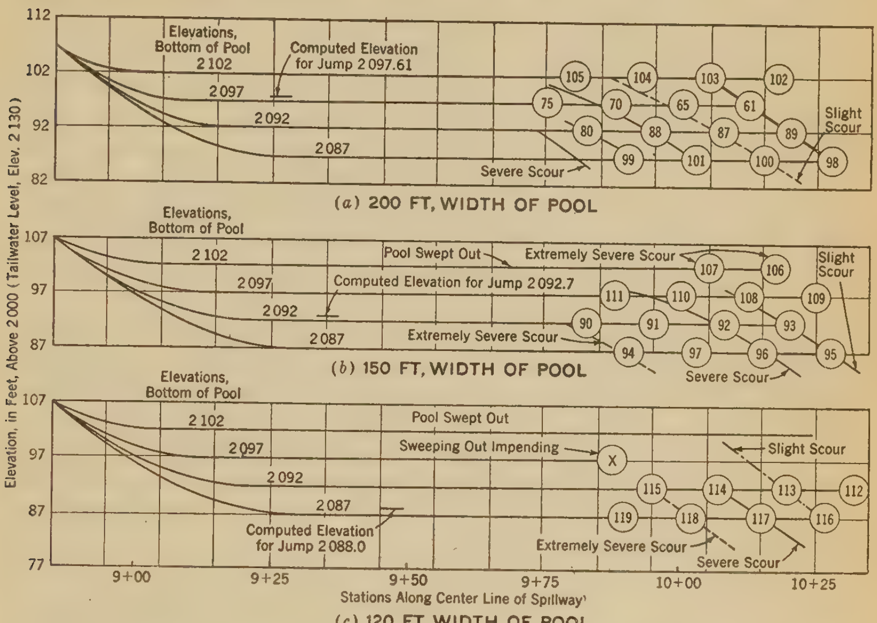
[5] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 527.

A comparison of the tests is given on Fig. 14, in which the various widths, depths, and bottom elevations are shown, and lines representing equally severe scour on the channel sides are drawn. The tests numbers are in circles, and the location of the circles indicates the end of the pool floor in that test. The results indicate that the severity of scour increased as the pool length was decreased, but that the scour was not decreased by deepening the pool. The former result might be reasonably expected, but a decrease of scour with increased depth seemed probable, due to the greater volume of water in the pool with the greater depth and, therefore, a greater dissipation of the energy.



(*a*) 200 FT, WIDTH OF POOL

(*b*) 150 FT, WIDTH OF POOL

(*c*) 120 FT, WIDTH OF POOL

FIG. 14.—COMPARATIVE SCOUR WITH POOLS OF VARIOUS LENGTHS, DEPTHS, AND WIDTHS

The results showed that for the same pool length, the severity of the scour was nearly independent of the depth as long as the jump remained in the stilling basin. This action is believed to be due to the more efficient formation of the jump with the smaller tail-water depth. With high tail-water levels the jump forms on the sloping floor leading into the pool. In such cases, the incoming water tends to dive under the water in the pool without causing a well-formed jump. As the tail-water level is lowered, the jump forms farther down the slope where the bottom is more nearly level; the tendency to dive under is less, and a more perfect jump is formed. This tendency to form a more perfect jump at smaller depths offsets the advantage in the greater depths, of the greater mass of water in which to dissipate the energy, with the result that the efficiency of the basin is independent of the depth, as long as the jump remains in the pool.

If the floor was raised too high, the depth was insufficient to cause the hydraulic jump in the stilling basin and the pool was swept out, as in Runs Nos. 106 and 107 (Fig. 14(*b*)). Such conditions produced severe scour on the sides of the earth channel down stream from the pool, but no severe scour on the stream bed. For each width, the elevation of the pool floor required to provide the theoretical height for the jump is indicated. It was possible to have considerably less depth than the jump theory indicated, without causing the pool to sweep out. Part of this difference may be due to the fact that the sills obstructed the flow somewhat and caused higher tail-water depths in the pool than in the stream below, where the tail-water elevation was observed. All the difference can scarcely be accounted for in this way.

The author obtained similar results, as is shown by the existence of many Type II conditions with values of $\frac{D_w}{D_j}$ less than unity. In the design of the stilling basin for the Cle Elum Dam, the full theoretical depth was provided, as a factor of safety, to take care of bed retrogression or other contingencies. The length of the pool adopted was that giving a "a slight scour" condition as indicated by the author's Fig. 1, and rip-rap was provided to protect against the scour. Economic rather than hydraulic considerations led to the selection of the 200-ft width for the prototype. These three requirements were met by a pool 100 ft long, 200 ft wide, and a floor 33 ft below the tail-water level for 40 000 sec-ft discharge.

When the dimensions of pools producing slight scour are expressed in terms of $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, and plotted as in the author's Figs. 7, 12, and 13, they indicate lines roughly parallel to the inclined portion of the solid line on Fig. 7, which divides the satisfactory from the unsatisfactory cases, but they give lower values of $\frac{L}{D_w}$. For the experiments with the 200-ft bottom, the corresponding values of $\frac{L}{D_w}$ are only two-thirds as great as the author's Series *A*. For the experiments with the 150-ft and 120-ft widths they are even less, being only 58% for the latter width. There are several factors which may account for this difference. One is that the incoming slope for the Cle Elum experiments was 1 on $1\frac{1}{2}$ as compared with 1 on $\frac{1}{2}$ (approximately) in the author's experiments. The flatter slope would probably give a more efficient hydraulic jump. Furthermore, on the flatter slope, the jump would begin farther up stream from the point where the pool was assumed to begin, which was at the intersection of the incoming slope with the pool floor, and, therefore, for a given nominal pool length, the effective length would be greater for the flatter slope. A third factor is that in the Cle Elum type the pool may continue in effect beyond the end of the floor, as the sill does not obstruct the flow, but only protects the bed from scour.

In order to make the results of the Cle Elum tests as readily applicable as possible to other conditions, Fig. 15 has been prepared. It is based on the assumption that the distribution of discharge in the experiments was uniform across the entire pool, and on the theory of model similitude. The foregoing assumption is believed to be substantially true. Suppose, for example, it is desired to design a spillway for a discharge of 25 000 sec-ft and a fall of 80 ft (plus friction loss) from reservoir level to tail-water level. The velocity head of the water entering the stilling basin, at the elevation of the tail-water, would then be 80 ft. Entering Fig. 15 with this value gives a model ratio of 43. This means that if the model experimented on had been considered to be a 1:43 model (instead of a 1:50 model as was required, in order that it meet the conditions for the Cle Elum Dam), the proportions would be correct for a velocity head of 80 ft.

Since, by the model theory, the form tested would represent a 1:43 scale as accurately as the 1:50 scale, the model would represent reliably the desired conditions as well as those at Cle Elum Dam. With a model ratio of 43, the lines representing discharge per foot of pool width show 135, 170, and 220 sec-ft per ft of width as determined from the experiments with model pool widths of 4.0, 3.0, and 2.4 ft, respectively. For a discharge of 25 000 sec-ft, a pool 114 ft wide would be required if the discharge was 220 sec-ft per ft of width as determined from the 2.4-ft model pool. The line representing depth of pool tail-water level based on the 2.4-ft pool width shows for a 1:43 model, a depth of pool below tail-water level of 36 ft, and the similar line for length of pool required, for a 1:43 model gives 99 ft. Since the sill height of the model was 0.2 ft, that required would be $0.2 \times 43 = 8.6$ ft high. Thus, Fig. 15 shows that a pool, 114 ft wide, 99 ft long, and 36 ft deep, with a sill 8.6 ft high, would meet the required conditions.

Other pool dimensions equally suitable, could be obtained from the lines representing the results determined from the 3.0 and 4.0-ft model pools. Of these three sets of dimensions, the most advantageous from the standpoint of cost or other considerations, may be chosen. Care should be exercised, however, to use throughout the dimensions determined from the lines based on the same model pool width, and not to use, for example, the depth obtained from the 4.0-ft model pool and the length as indicated by the 3.0-ft pool. It is recognized that the dimensions indicated by Fig. 15 may not be very practical in certain cases. This simply means that the Cle Elum model tests are not applicable to those cases, and data must be sought elsewhere for their design.

In applying this diagram the following conditions to which it is applicable should be kept in mind: The flow must be equally distributed across the stilling pool and in the direction of its axis. The tail-water rating curve at the site must be such that ample pool depth is available at all discharges to cause the jump to form in the basin. The top of the sill must be sufficiently below the channel bottom so that a drop of the water level will not result at any discharge from the obstruction caused by the sill. There will be considerable wave wash on the sides down stream from the pool, and rip-

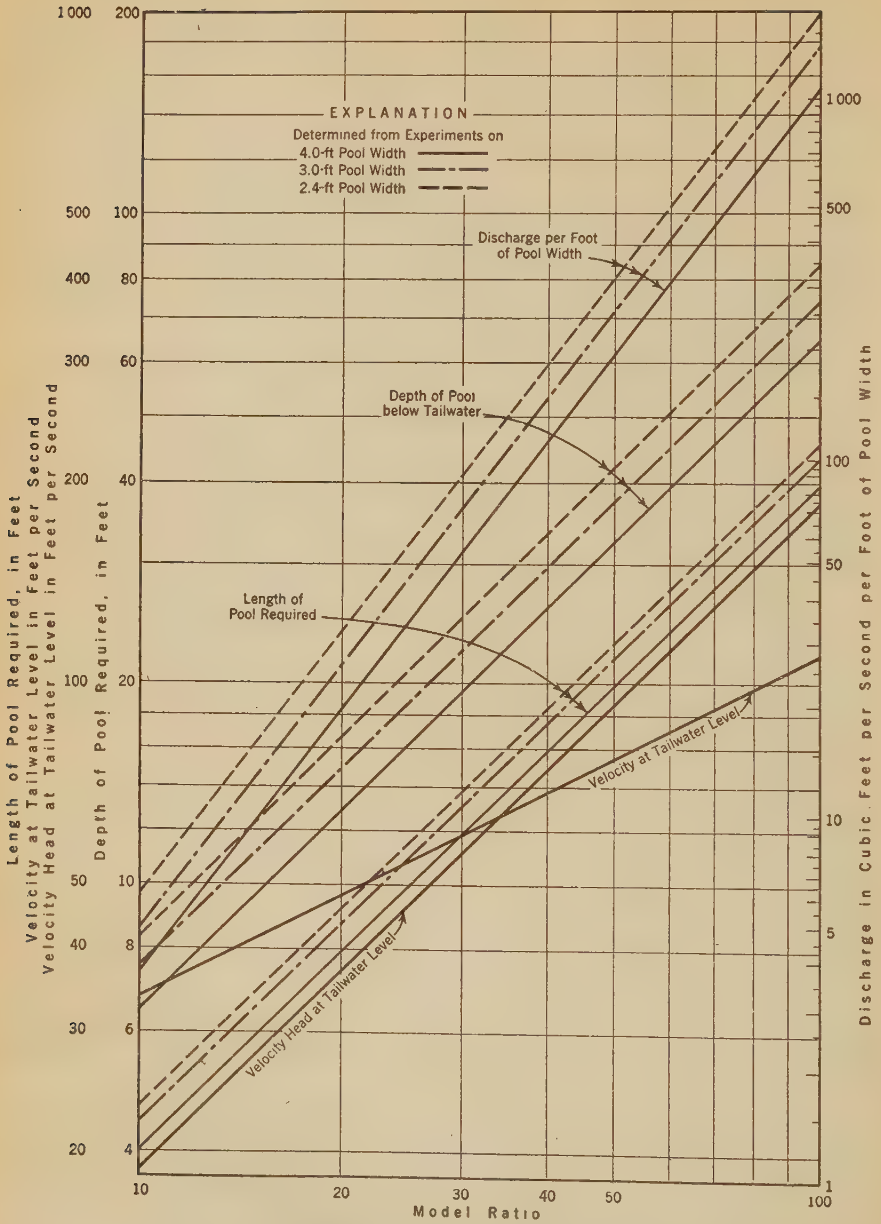


FIG. 15.—DIMENSIONS REQUIRED FOR STILLING POOLS AS SHOWN BY RESULTS OF MODEL TESTS

rap protection will be required, especially if the pool is intended to be used over long periods at flows approaching its capacity.

Although the experiments were performed with a dentated sill it is believed that the ogee or beveled sill types could be used with slightly less desirable results. The experiments were performed with a slope of 1 on 1.5 entering the pool and a transition curve at the bottom of the slope. It is believed that flatter slopes with the transition curve would produce equally as good results, but that the effects of steeper slopes might be less desirable. The writer agrees heartily with Mr. Stanley that "the day is not far distant when experiments with models will be a necessary part of the design of every overflow dam." The data given herein, therefore, should not be considered as removing the necessity of model tests, but only as the basis for preliminary designs, to aid in narrowing the field to be covered by the model studies, and for the design of unimportant structures of insufficient magnitude to bear the expense of model tests.

C. C. INGLIS,[6] ESQ. (by letter).—For the very narrow limits of the experiments—namely, two-dimensional parallel flow, horizontal bed, and free flow-away down stream—stilling basins of Types I and II give highly satisfactory results; but this is only a special case of a very great problem. More generally, dissipation of energy is a three-dimensional problem, and eddies with vertical or complex axes cause much damage. Where energy has to be dissipated and eddies have to be prevented in canals, a combination of a "baffle" and a "deflector" has given satisfactory results in Sind.[7]

The baffle is made equal in height to the free-flow depth and is constructed at a distance from the toe equal to $5\frac{1}{4}$ times the free-flow depths. The baffle is fixed at such a level that, with the natural down-stream water level, the wave will form at the toe of the fall—under which conditions the standing wave dissipates the maximum amount of energy. The deflector is fixed at the end of the pavement, and is equal in height to one-twelfth the maximum depth of water.

The falls and regulators of the Sukkur Barrage Canal in Sind, have been designed as proportional meters. This involves fluming down to about one-half the natural bed width. The convergences have been standardized at 1 in 5 and the divergences at 1 in 10, although sharper divergences have been used in several cases. Under such conditions the corkscrew eddies, which form near the banks below falls, are accentuated and—especially where the divergences are sharper than 1 in 10—cause scour at the sides down stream.

Such scour, if not severe, can be prevented by '"tarungars"—wire crates containing stones—which should be laid sloping downward at a slope of 1 on 5. A neater solution is to use deflector-grids; but perhaps the best solution is to lower the pavement, joining it to the side slopes by elliptical curves with the deflector continued up the side slopes.

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[6] Superintending Engr., Irrig. Development and Research Circle, Poona, India.

[7] "The Dissipation of Energy Below Falls," by C. C. Inglis and D. V. Joglekar, *Technical Paper No. 44*, Public Works Dept., Bombay, India.

Where the divergences were sharper than 1 in 10, it was found necessary to fan out the flow to prevent return flow. This was effected by building vanes in continuation of the piers, the vanes being splayed as found best in model experiments.

The results obtained with baffles, deflectors, and vanes have proved highly satisfactory, whereas several other methods—including the one described by the author—when tested, using large models, gave much inferior results.

F. KNAPP,[8] ESQ. (by letter).—The author deserves credit for the experiments made to rationalize the design of stilling basins at the toe of an overflow dam. Especially valuable are Figs. 7, 12, and 13, which show the relationships between the fundamental and controlling factors. Mr. Stanley classifies five distinct types of stilling-basin action, Types I and II (the most efficient) being identified as roller and jump action, respectively. The writer does not feel satisfied with the distinction between these types. From the description given under the heading "The Stilling Basin," Type I can also be considered as jump action with the nappe submerged by tail-water. Type II represents free nappe with repelled jump. According to B. A. Bakhmeteff,[9] M. Am. Soc. C. E., the ratio,

$$\frac{\text{Difference of elevation, between head and tail-water}}{\text{Height of dam}}$$

defines the two types of jumps. Type II requires a ratio > 0.75 and Type I, a ratio < 0.75 approximately, the correct value depending on the over-flow conditions of the dam. Experimental observations by Bazin confirm Professor Bakhmeteff's deductions.

Under the heading "The Results, Series *A*", Mr. Stanley states that the action classified as Type II occurs when the ratio, $\frac{D_w}{D_j}$, is between 0.85 and 1.15, or substantially equal to unity, and the ratio, $\frac{L}{D_w}$, exceeds about 4.5. In other words, the depth of the pool must be that required for the formation of a hydraulic jump and the length of the pool must not be less than about 4.5 times the depth of the jump. There are cases, however, that require a ratio, $\frac{L}{D_j}$, considerably greater than 4.5. Fig. 7 shows this ratio varying from 4.6 to 7.2 for Type II action and a ratio, $\frac{D_w}{D_j}$, equal to unity. With the length of the pool determined, either theoretically or empirically, the problem of designing a stilling pool for Type II action, would be solved. This length may be determined by the following considerations.

The loss of energy in a hydraulic jump is always smaller than the Borda-Carnot loss, $\frac{(V_1 - V_j)^2}{2g}$. A certain distance is required before shooting water in the jump can be slowed down to normal, flowing water. Consequently, with the velocities changing gradually, the loss of head produced by the jump must be smaller than $\frac{(V_1 - V_j)^2}{2g}$. In other words, there must be a relation between the length of the jump, L, and the loss of head in the hydraulic jump, namely $\left(d_1 + \frac{V^2_1}{2g}\right) - \left(d_j + \frac{V^2_j}{2g}\right)$. Based on published experiments[10], the writer has deduced the following empirical formula (in metric units) for the length of the jump:

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[8] Designing Engr., The Sao Paulo Tramway, Light & Power Co., Ltd., Sao Paulo, Brazil.

[9] "Hydraulics of Open Channels," *Engineering Societies Monographs*, New York and London, 1932.

$$L = \left(62.6 \frac{D_1}{H_1} + 11.3\right) \left[\frac{(V_1 - V_j)^2}{2g} - (H_1 - H_j)\right] \dots\dots\dots (2)$$

in which, $H = d + \frac{V^2}{2g}$.

Unfortunately, it is not possible to present a comparison of this formula with the results obtained by the author. The writer is unable to check the depth, D_1, of the jump from the values of discharge and the depth, D_j, given in Table 1. The writer hopes Mr. Stanley will clarify this point in his closing discussion.

C. MAXWELL STANLEY,[11] JUN. AM. SOC. C. E. (by letter).—Discussions of this subject served to emphasize certain parts of the paper and to raise certain questions regarding the applicability of the methods of design suggested therein. The writer wishes to express his appreciation of these contributions.

The discussion, by Mr. Lane, of the test data from the Cle Elum Dam of the U. S. Bureau of Reclamation is particularly interesting. The results of these experiments in stilling-basin design check closely the results submitted in the paper. Mr. Lane points out that in the model study of the stilling basin for the Cle Elum Dam, the results parallel the inclined position of the solid line on Fig. 7, which divides the satisfactory from the unsatisfactory cases, but that lower values of the ratio, $\frac{L}{D_w}$, were obtained. He suggests three factors which may account for this difference: (1) The flatter incoming slope which would allow a more efficient hydraulic jump; (2) the fact that the jump would begin up stream from the point at which the pool was assumed to begin; and (3) the effect of the Cle Elum type of pool may continue beyond the end of the floor as the sill does not obstruct the flow but only protects the bed of the river from the scour.

There is little question but that the flatter incoming slope will allow the formation of a more efficient jump. This fact was encountered in the writer's experiments and in the analysis of the effect of the shape of the dam he indicated the possibility that a flatter slope would allow the jump to form more easily, thus decreasing the length of basin needed for the complete formation of the jump. The fact that the jump will form on a slope also produces the effect of requiring a shorter stilling basin. The third condition (the possibility of action continuing below the end of the pool), is related to the effect of submergence upon the weir. As mentioned in the paper, experiments were run with the weir submerged and while these experiments were not sufficiently extensive to permit any complete analysis, it was very apparent that with a depth of water below the weir sufficient to submerge it, the capacity of a given stilling basin to dissipate energy was increased.

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[10] "Untersuchungen über den Wechselsprung," von Safranez, *Der Bauingenieur*, 1929, No. 38, p. 676.

[11] Engr. (Young & Stanley, Inc.), Muscatine, Iowa.

The writer would like to suggest a fourth reason why the results of the Cle Elum tests indicated satisfactory operation at lengths somewhat less than those found in his own experiments. In the latter experiments satisfactory operation was separated from unsataisfactory operation merely by observation of the flow over the weir. If this flow was smooth, indicating that turbulence had been removed, the action was classified as satisfactory. It was hoped to supplement this with further experiments using an erodible bed below the weir. It is the writer's opinion that it is unnecessary to remove all the turbulence from the water prior to passing over the weir in order to avoid erosion below it. This should be particularly true with submerged conditions similar to those used on the Cle Elum test.

The effect of the weir is twofold: First, it creates a certain pool depth, and, second, it acts as a sill to create an eddy, or roller, about a horizontal axis immediately down stream from the weir. The direction of this roller is such that the water in contact with the bed of the river flows up stream toward the sill and thus prevents erosion. This phenomenon has been observed in numerous model studies, and advantage has been taken of it in the design of many protective devices below spillways.

Mr. Inglis questions the applicability of the data to any except a few special cases. He brings out the fact that dissipation of energy is a three-dimensional problem. This is true, but if the spillway, or stilling basin, is of any considerable length, the eddies or currents at the side will affect only a limited length of the basin, and the major part of the design must be based on the two-dimensional condition similar to that studied. Proper care must be taken of the action of the side currents and eddies. Such actions will be special cases for every project as they are concerned with local topography to a considerable extent.

Mr. Knapp does not feel satisfied with the distinction between Types I and II and suggests that Type I can be considered as jump action with the nappe submerged by tail-water, while Type II represents free nappe with repelled jump. The difference in action between Types I and II, as shown in Figs. 1 and 2, is very distinct, and there was nothing in the appearance of the action to suggest that Type I is a form of jump action. It is merely a stream of water shooting into a pool, flowing along the bottom of the pool, and, at the same time, creating a roller or eddy about a horizontal axis.

One point, which has been stressed by Mr. Lane and also by the writer, is worthy of further mention. This is the fact that the design of a stilling basin for any particular project is not only a matter of hydraulics, but also one of economics. For any condition there will be a number of combinations of length, depth, and submergence which will dissipate satisfactorily the energy from the falling water, and the choice of a particular design will depend on the relative costs. Hence, it is an economic problem.

In conclusion, the writer wishes to emphasize again the need of additional research to determine more clearly a rational method of design of stilling basins. The need of research on the effect of different slopes of the dam and upon the submergence of the basin is particularly apparent from the tests on the Cle Elum Dam.

If this paper serves to stimulate further research in this field, it will indeed have served its purpose.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1868

A PROBLEM OF SOIL IN TRANSPORTATION IN THE COLORADO RIVER

By S. L. Rothery,[1] M. Am. Soc. C. E.

With Discussion by Messrs. C. E. Grunsky, Ivan E. Houk, H. M. Rouse, William T. Collings, Jr., R. F. Walter, Harry F. Blaney, Calvin V. Davis and Dirk A. Dedel, Thaddeus Merriman, J. C. Allison, and S. L. Rothery.

Synopsis

This paper is concerned principally with the problem of excluding soil, transported by the Colorado River, from the Canal System of Imperial Valley, in California. Its purpose is: (*a*) To feature the relative importance of the bed load of the Colorado River and to provide some conception of the extent of the unknown volumes of soil transported; (*b*) to present known and anticipated future river-flow conditions pertinent to soil conveyance; and (*c*) to propose fundamental requirements for a diversion structure that will exclude all the bed load and a part of the suspended load, thus permitting minimum sizes for—or, perhaps, eliminating as unnecessary—the enormous settling basins which are expected to desilt the large diverted flow, and also permitting lessened sluicing operations for the disposal of the sludge.

The publication of specific research and a pooling of available knowledge before a design for such an important diversion is finally adopted, may suggest a solution for the automatic exclusion of much of the soil load. Each 1 000 000 cu. yd. of soil that is prevented from crossing the sill of the headgate, can represent an annual saving of tens of thousands of dollars if relief must be obtained by dredging. With 10 000 000 cu. yd. of soil automatically excluded, such savings are probably in the hundreds of thousands of dollars, depending on the remedial measures that have to be taken when such large

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Note.—Published in December, 1932, *Proceedings*.

[1] South Pasadena, Calif.

volumes of transported soil are continually arriving on the down-stream side of the diversion gates. In part, these savings are represented in the form of lessened maintenance operations throughout a vast canal system, in channels and sinks for waste-water disposal, as well as in the ditches of the individual ranches.

INTRODUCTION

As a background for a thorough understanding of the problem reference is here made to a paper on the subject by the late C. E. Grunsky, Past-President, Am. Soc. C. E., published in 1929.[2] As they apply to the intended purpose of this paper, the following items of information from that paper are summarized:

(1) The water supports and carries its suspended load throughout the entire length of the Imperial Canal. Sometimes there is less, and sometimes a little more, sediment in suspension at the head of the canal than at points 20 to 40 miles below but, generally, when the water is extremely muddy at the head it stays muddy throughout the entire length of the canal.[3]

(2) A phenomenon that illustrates the occasional conditions favoring the temporary conversion of the bed load of a river into a suspended load is the formation of a standing wave. This is noted in streams of moderate depth, flowing in beds with fine sand bottoms. It is a common phenomenon on the Lower Colorado River.[3]

(3) In 1918, the head-works of the canal were modified, suction dredges were installed, and, in a period of 30 months, nearly 15 000 000 cu. yd. of material was excavated. While it is not quite clear what effect this had on the lower reaches of the Imperial Canal, it is certain that if it had been allowed to remain in the canal, it would have traveled down stream to the annoyance of the Canal Management and the farmer.[4]

(4) The dredging did not relieve the canal of its bed load entirely. The dredged sumps in the head of the canal did not trap all the silt that came over the flash-boards of the Rockwood Gate. Even 500 000 cu. yd. per month does not represent the entire load of fine sand carried as a bed load by the canal. How much more was being transported is not known, and no entirely satisfactory basis for estimating it has been found.[4]

(5) Assuming that there must have been some débris and other material held in suspension temporarily during ordinary flow conditions, the bed load of the Colorado River (with its annual discharge of about 15 000 000 acre-ft. of water at Yuma, Ariz.) should be about six or seven times as great as that of the canal, or 25 000 to 30 000 acre-ft. per year.[4]

(6) Observations at Yuma in 1914 indicated that when the Colorado River was heavily charged with silt, the settling was nearly always slower than when the silt load was light. The Imperial Canal does not hold quite

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[2] "Silt Transportation by Sacramento and Colorado Rivers and by the Imperial Canal," by the late C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1104.

[3] *Loc. cit.*, p. 1113.

[4] *Loc. cit.*, p. 1117.

as much silt in suspension as the river. The water in the canal is less turbulent and the heavier grains of silt drop down and become part of the bed load.[5]

(7) Turbidity in the river varies with depth. The increase of the suspended load downward from the surface bears no fixed relation to the loads at the surface or at mid-depth. The increase with depth is nearly constant in the sense of being independent of the total suspended load, the increase of percentage being 0.11 from the surface to the bottom. In other words, if the percentage by weight at the surface is represented by P, that at the bottom is $P + 0.10$.[6]

(8) The Colorado River is not as muddy at low stages as at moderately high stages, as, for example, a discharge of 15 000 to 50 000 sec-ft. The river is muddiest when it is discharging 15 000 to 30 000 cu. ft. per sec. Turbidity is highest in the early spring and in the fall.[7]

(9) Perhaps there are other causes affecting the degree of muddiness. For example, the silt in the lower river shows many signs of being dispersed, that is, the colloidal particles seem to be carrying negative charges. Possibly, the source of the water and its chemical composition affect the degree of dispersion, or of flocculency, as the case may be, and, therefore, cause a greater or smaller volume of silt in suspension. When clays are dispersed, each flocculated particle splits into many subdivisions, sometimes so small as to be colloidal.[8]

(10) Table 11[9] of Mr. Grunsky's paper shows that in periods, such as October to December, 1925, and January, February, June, and July, 1926, the suspended load increases with the distance below the Grand Canyon; in other periods, it remains fairly constant. In September, 1926, there was even a decrease of suspended load. These results seem to confirm the conclusion that, at times, the river picks up material from its bed and, at other times, it drops part of its suspended load.[10]

(11) The facts illustrated in Mr. Grunsky's Fig. 6[8] give no indication that can be used in estimating the bed load of the river, except possibly that a considerable portion of it originates below the Grand Canyon.[11]

The Bed Load

An average of 15 000 acre-ft. of silt, the equivalent of 24 200 000 cu. yd., has been diverted annually in suspension into the Canal System of Imperial Valley, California. In 1923, the peak quantity was 24 900 acre-ft., or 40 172 000 cu. yd. In addition to this quantity, large volumes of bed sand and coarse silt, estimated at more than 4 000 acre-ft.,[4] or 6 453 000 cu. yd., have been transported as the annual bed load along the Main Canal. The respective annual quantities transported on the Lower Colorado River are estimated to be: 94 000 to 111 000 acre-ft. of suspended soil, and 20 000 to 30 000 acre-ft. of bed load.[8] These estimates of the bed load are believed to be much too small.

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[5] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1122.

[6] *Loc. cit.*, p. 1127.

[7] *Loc. cit.*, p. 1129.

[8] *Loc. cit.*, p. 1130.

[9] *Loc. cit.*, p. 1131.

[10] *Loc. cit.*, p. 1132.

[11] *Loc. cit.*, p. 1133.

At a great diversion works, 19 miles up stream from Yuma, provision is being made (1932) for a maximum inflow into a main canal of two to two and one-half times that of the present summer irrigation demand. Apparently, this should make possible, the doubling of each of the foregoing two soil quantities in the main canal if the intercepting influence of the huge reservoir formed by Hoover Dam could be excepted. A cost of $3 375 000 is listed tentatively for the construction of desilting works at the proposed site.

That the suspended load of the Colorado River, as now accepted,[12] is only a part of the total soil annually transported, is logically shown by the following reasoning applicable to the spring flood each year, as well as in a lesser degree when smaller floods or freshets occur. Consider, for example, the 300-mile stretch up stream from Yuma. The bed is filled with soil subsequent to several winter months of low-river stages; then 4 to 6 weeks of increasing flow to the flood peak in May or June completely removes a solidity of soil represented by the product of 300 miles, an average scoured depth of, say, 9 ft., and a river width of, say, 600 ft. Any material in this space is influenced by fluid velocities of 4 to 10 ft. per sec., and it is, therefore, impossible for deposition or up-stream travel of any sand or silt particles to refill the scoured prism before the discharge is in the decreasing stages. This product is equivalent to 316 800 000 cu. yd., or 196 400 acre-ft. of soil completely removed and transported hundreds of miles in a period of only a few weeks. On the other hand, the estimated soil volume carried in suspension averages about one-half this quantity for an entire year!

The scoured deepening is often more than 20 ft. At Yuma in 1907 and, again, in 1909, it was found that for an increase in the gauge height the bed was lowered approximately 30 ft.[13] River widths of flood stages are between 800 and 1 200 ft., and the time of travel from the Grand Canyon to Yuma is about 4 days in high-water stages. Therefore, the values given for the elongated prism of removed soil are conservative, and the entire length is at a scoured maximum simultaneously during the few days of peak flow.

The additional square feet of scoured area of cross-section probably cannot be expressed in a percentage of the water volume as solid material in motion, since the distance to deposition and the time unit of the flood duration do not seem to permit of a constancy of expression.

The velocity curve of a stream in a vertical plane is represented by the line, *A B*, in Fig. 1, the maximum velocity being in the upper half of the vertical depth, and the minimum at the stream bed. The fine silt particles that aggregate the "suspended" soil load, and that are responsible for the turbidity of the stream, are buoyed up and transported in the upper two-thirds to three-fourths of the stream depth with less effort on the part of the stream, than in the lower depth, while general uniformity of flow is maintained, as is the case of an irrigation canal whether it is 1 mile or 300 miles in length.

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[12] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1142.

[13] *Loc cit.*, Vol. LXXVI (1913), p. 1214.

Particles too heavy to rise, or too granular to be retained in suspension (unless turbulence is produced by some cause to destroy uniformity of flow) are transported by rolling and saltation along the bed by the lesser velocities, which, however, are not much less than those above the center depth. The velocity of flow necessary to move fine sand is 0.70 ft. per sec., and, for coarse sand, 0.80 to 1.00 ft. per sec.[14] The mean velocities for flow conditions in the channels and canals of the Colorado Delta region are 3 to 6 ft. per sec.; so that with bed velocities known to be greater than 1.00 ft. per sec. in the more slowly moving canal water, when flood conditions exist in the river the travel of the bed load is both rapid and continuous.

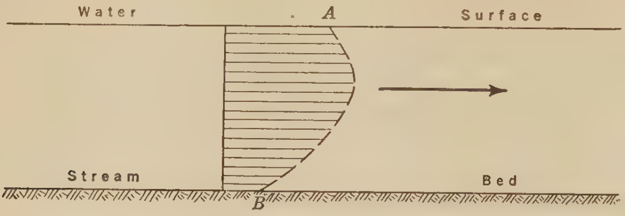

FIG. 1.

Some samples of the traveling bed load were collected at Grand Canyon from 1925 to 1928. When the sampling bottles were near the bottom with the mouths pointed up stream the silt content was 35% greater for a mean of sixteen samples, than when the samples were taken in the usual upright position. For six samples taken in a similar manner, but still nearer the bottom, the silt content was 40% greater.[15] Such increased percentages are properly part of the unknown bed load, if only for the reason that they have never been considered in estimating the suspended loads. This also shows that there is a traveling slurry of soil contiguous to the bed at normal river stages.

Consider such a fluid slurry in the lowest, say, 3-ft. depth of a large flood. Bed sand itself has no cohesive qualities, but it can have compactness when the interstices are filled with fine silt and when there is a weight due to depth of water above. The drag of increasingly scurrying velocities over it frets away the compacted particles, giving them initial movement and lessened weight due to becoming freely submerged. The loosened grains soon attain momentum, the velocity of which approximates that of the fluid influence surrounding them. This surely must be true for sand, since egg-sized pebbles are in motion with 3-ft. velocities, and boulders, 6 to 8 in. in diameter, are moved when velocities of 5 to 6 ft. per sec. are attained, and this latter rate is only the average mean velocity of Colorado floods.

If the sand load thus being shot along in the bottom 2 or 3 ft. of the flood, were represented only by a 3-in. average thickness of solidity for a 600-ft. width of river and averaging a velocity at the bottom of only 3 ft. per sec., the bed-load volume would be 17 cu. yd. per sec. at any given point, which is equivalent to 1 440 000 cu. yd. per day, or 60 000 000 cu. yd. in the 6 weeks of rising flood. Visualize this soil supply as coming from not any one given location, but from the continued welling up of the bed into fast fluid motion along hundreds of miles of this elongated soil prism. The 3-ft. velocity represents, say, 50 miles per day. Then, in a 300-mile length there would be a daily feeding of sand to the traveling load of 1 440 000 × 300 ÷ 50 = 8 640 000 cu. yd., which, if continuous for 6 weeks, or 42 days, becomes 360 000 000 cu. yd., or 223 000 acre-ft. of soil.

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[14] "The Control of Water," by the late Philip A' Morley Parker, M. Am. Soc. C. E., p. 495.

[15] *Water Supply Paper 636-B*, U. S. Geological Survey, p. 26.

In the first flood stages, at 30 000 sec-ft., the traveling thickness of equivalent solidity may be, say, less than an inch, and at the flood peak it may be the equivalent of feet in thickness. Any value used must necessarily be an assumption. The writer is of the opinion that 3 in. as an average is too conservative, and that scour is a continuous process of excavation in a great depth of alluvium, until flood velocities have passed. If the discharge lessens temporarily, the water surface lowers; but scour and the rate at which sand travels are functions of the flood velocities which are maintained for some days after the flood peak has passed.

Assuming that the bed of the river is of alternate "deeps" and "shoals," so that it may be claimed that the effective length is reduced 50%, which would divide this great scoured volume in two, the result would still be greater than that of the estimated suspended load per year. This soil movement has been accomplished in six weeks from only a part of the channel.

A conspicuous example of the power of the river to excavate soil was described in a previous paper[16] wherein the writer stressed the transportation of such soil volumes in a unit length of 100 miles.

Anticipated River Flow Conditions Below Hoover Reservoir

The average annual run-off of the Colorado River is 15 000 000 acre-ft., with years of maximum precipitation yielding more than 20 000 000 acre-ft. The planned storage capacity of Hoover Reservoir is 25 000 000 acre-ft. Until large agricultural uses are developed up stream from the reservoir to consume the water, the river bed below the huge dam, in consecutive wet years, may have to take care of a flow nearly equal to the annual run-off. A regulated continuous flow of 21 000 sec-ft. throughout the year would provide this requirement; but, because of the necessity of conserving head at the dam for development of power, such steady flow will never occur, and the regulation of discharge will depend upon the down-stream uses, and upon the estimated up-stream flood inflows as to their effect on the surface elevation of the conserved water in the reservoir.

Down-stream uses are a minimum in the winter months. With a release of 15 000 sec-ft. for six months to provide these uses, including the quantity required for sluicing at the Imperial Diversion, 5 500 000 acre-ft. will have been discharged from storage by March, and this volume in a wet year may have been more than regained by inflow, so that the reservoir has been maintained as nearly full. The spring flood with 9 000 000 to 10 000 000 acre-ft.

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[16] *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1412; also, *Engineering News-Record*, Vol. 95, (1925), p. 1068.

in 60 days (as in May and June, 1917, 1920, and 1921) would demand a release of 83 000 sec-ft. While this discharge will undoubtedly be reduced in the demand for regulated down-stream control, it is to be expected that the range of flow will be between 15 000 and 65 000 sec-ft. in the lower river after the Hoover Reservoir is filled.

The stream will be a different and chastened river, however, when the Hoover Dam is placed in operation. Although the river issues from the dam as clear water without its up-stream soil loads, 300 to 450 miles of soil-filled river bed are ahead; but its capacity for deep scour and for refilling with soil its channelized bed will have been greatly curtailed.

Since the profile gradients have resulted from the necessary velocities for silt-laden water in the alluvium, they will not be stable enough to produce and maintain the lesser velocities needed for the flow of clear water. The required gradients will be less steep and the stream, in striving to attain these new slopes, will continue to pick up soil loads, because the moderately high river stages are capable of transporting a maximum turbidity. The references (Items (8), (10), and (11)), to highest soil loads for discharges of 30 000 to 50 000 sec-ft., have their verification in studies by Raymond A. Hill, M. Am. Soc. C. E., from 3 000 samples taken between 1908 and 1916 at Yuma.[17] Mr. Hill calls attention to the fact that the characteristics of a desilted river cannot be based on data acquired when the river was heavily laden with silt and that no one can predict which rate of discharge from a flood-control reservoir will produce the most stable condition.

In commenting on the maximum silt content at discharges of 40 000 sec-ft., E. W. Lane, M. Am. Soc. C. E., states[18] that the redistribution of flow might even increase the silt removal to more than 80 000 acre-ft. per year.

Eventually (after several years), the required river gradients for clear water flow will become established through gradual recession working up stream from control locations in the bed, such as from the diversion dams of the Imperial Intake and of the Metropolitan Water District at Parker, Ariz.; and also from locations where rock or hard soil is close to the river bed. With varying discharges (the maximum being four to five times as great as the minimum), erosion, deposition, and meandering will still occur, although to a much lessened extent than with the extremes of flow of the unregulated river.

Tributary streams, notably the Williams River, will frequently contribute sudden freshets transporting coarse soils and sand, because the declivities of these side creeks and washes are relatively great. That of the lower end of the Williams River is more than twice the stream gradient of the Colorado for a distance of 10 miles from their confluence, and the high discharges are given as 7 000 to 9 000 sec-ft.[19] On August 5, 1931, a cloudburst on the Williams water-shed produced an increase of discharge from 7 000 to 45 000 sec-ft. over night.[20]

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[17] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 419.

[18] *Loc. cit.*, p. 418.

[19] *Water Supply Paper 409*, U. S. Geological Survey.

[20] *Los Angeles Times*, August 7, 1931.

It is concluded, therefore, that both suspended and bed loads will be present in the curtailed river flows below the Hoover Reservoir. The suspended loads, however, are expected to decrease in extent to almost negligible quantities as the regimen of the stream becomes stabilized. Bed sands are expected to be present always, although in much lessened quantities when the channel has attained equilibrium.

The physical difference of the "suspended" silts and the "bed" silts has been described[21] concisely by Harry F. Blaney, Assoc. M. Am. Soc. C. E. He distinguishes between the mechanical analyses given in Table 19 and in Table 1 of Mr. Grunsky's paper, previously mentioned, to the effect that 73% to 95% of the "suspended" load was fine enough to pass a No. 200 sieve, but that only 1.7% to 34% of the canal "bed" deposits will pass such a sieve in which the separation of the meshes is 0.10 mm. From these respective tables it is noted that the coarsest "suspended" silt passes a No. 40 sieve (wherein the meshes are 0.48 mm.), and the coarsest canal "bed" silt or sand passes a No. 20 sieve (wherein the meshes are 0.95 mm.).

The great bulk of the suspended load of the present turbid flows, therefore, is seen to be of the finest soil particles, but the proportion of such particles in the alluvial soils of the river bed, resulting from recurring deposits of each receding flood, is undoubtedly only a small proportion of the bed soils.

Charles Terzaghi, M. Am. Soc. C. E., refers to a property of silts as being scale-like and fragile (see, also Item (9)), always sinking in stilled water with the flat side horizontal, whereas sand grains are of rounded and angular shapes. His analyses[22] afford an explanation of what is perhaps the fundamental characteristic difference between the "suspended" and the "bed" loads of the Lower Colorado. It is readily conceivable that lamina and disk shapes are more sensitive to support and carriage in fluid motion than granular ones.

The sieve analyses mentioned, also show similarity of sizes of particles in both Tables 1 and 19 of Mr. Grunsky's paper previously noted, although the relative percentages of fineness and coarseness differ. As a result of silt research on the Nile River in Egypt,[23] "coarse sand" was classified as that collected on a No. 100 sieve; "fine sand" as that passing the No. 100 sieve, but caught on No. 200 sieve; and silt and clay as that passing the No. 200 sieve. The coarse sand in suspension in the bottom 3 ft. where the Nile is 26 ft. deep, is given by Mr. A. B. Buckley as 16 times the quantity of such sand in suspension in the surface 3 ft.; that for the fine sand as 7 times the respective quantity; while that for the suspended matter—silt and clay—shows little difference, the bottom quantity being 1.2 times that in the surface water. This is in close agreement with Mr. Grunsky's findings for the suspended silt content of the Colorado River (see Item (7)), which content is, however, ten times that of the Nile.[24]

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[21] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1137.

[22] *Engineering News-Record*, Vol. 95, p. 912.

[23] *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI, p. 205.

[24] *Loc. cit.*, p. 214.

Therefore, the clear-water flow from the Hoover Reservoir, in the erosive process of flattening the stream gradients, will absorb readily into suspension the finer silts to aggregate the capacity soil load. The granular sands and coarser silts will contribute in a lesser degree to this load, until such suspensible matter, entrapped in the alluvial deposits, has been released and transported. In other words, the silts (and turbidity) will be eliminated from the bed soils of the channel first, leaving the sands to continue to move by rolling and saltation after the clearness of the stream has become established.

Furthermore, sand will always be present, more or less, because each year tributary streams and washes that have steep gradients, will add supplies of sand and coarse soils transported by the cloudbursts that are a climatic feature of the territory. The largest of these streams are the Williams River (which divides into the Big Sandy River and the Santa Maria River), Sacramento Wash, Tyso Wash, Bouse Wash, and Arroyo Seco. Again, with the decreased flows becoming more uniformly regulated in distant years, scoured channelized depths will cease to exist, and the curtailed stream will tend to assume a wide shallow cross-section that is virtually all bed (see Fig. 2, in which the greater distance (*a*) is the width of channel for a stream the bed of which is in sandy soil, and the distance (*b*) is that width for an equivalent stream volume flowing in soils that have cohesive qualities, as in silts and clays). Because in such a stream, the relative area of the bed to the wetted surface is great, giving a large transporting width, sand travel may be expected to be a continuous feature of the flow; therefore, any structural provision effecting its exclusion from the diverted irrigation water will be permanently beneficial.

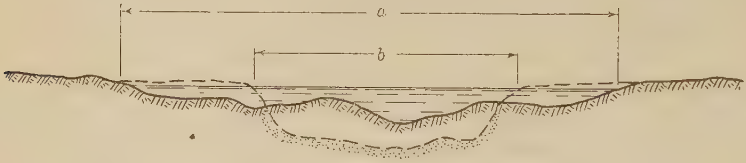

FIG. 2.

STRUCTURAL REQUIREMENTS FOR EXCLUSION OF THE BED LOAD

The quantity of sand and bed silt that has been passing through the headgate into the Main Canal is unmeasured. How much more than 500 000 cu. yd. per month (see Item (4)) is being transported is not known. Undoubtedly, the actual quantity has been more than twice the original estimate of 500 000 cu. yd. based on suction dredger yardage. The sumps to which Mr. Grunsky refers[25] for trapping the sand, are deep excavations in the bed of a canal more than twice too wide for the arc limits described by the suction head of the dredge swinging about a lowered spud. These sumps cannot be made too close to the canal banks. The traveling bed load fills the deep excavations rapidly as they are being made over, say, 50% of the bed width, while traveling past the dredger over the other 50 per cent. Shut-downs for

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[25] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1117.

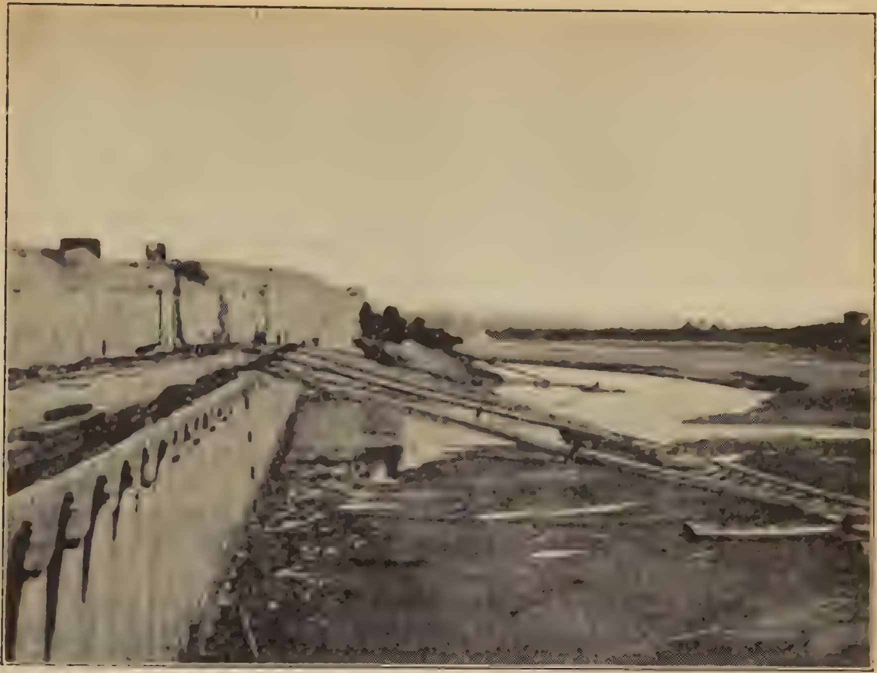

Fig. 3.—River Bed Flush with Silt for Extreme Low Water at Rockwood Intake.

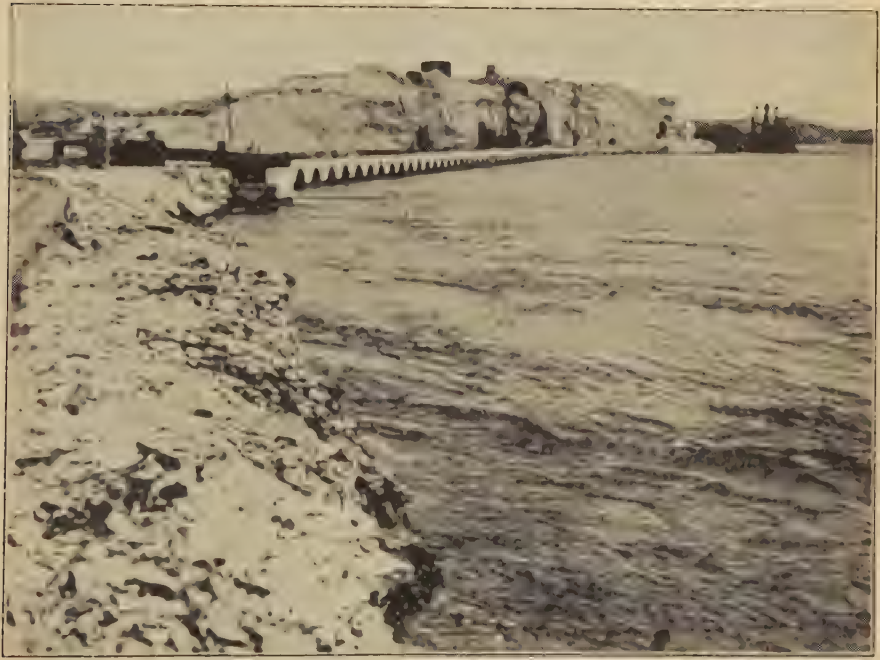

Fig. 4.—High Flood at Rockwood Intake.

repairs and while the dredger is advancing to new locations, represent time lost, giving the advantage to the bed load, the travel of which is ceaseless.

Fig. 3 shows extreme low-river conditions on the river side of the intake gate, the bed of the river being flush with the lowest sill of the structure. The "drag" on the bed in the direction of the intake is apparent. The structure is composed of seventy-six openings that invite the traveling bed load, except during flood conditions. Fig. 4 shows the flood conditions, during which the bed is probably at great depth below the sill, which is also raised several feet; but the sudden retardation of velocity of the canal inflow, contributes the bed load in the canal.

When extremes of discharge are removed, with scoured river conditions the head-gate sill should have a sluicing velocity below it, tending to sweep the traveling bed load past the structure without coming over the sill; but with a diversion taking the canal supply from a direction at right angles to the river, interference of uniform flow conditions (especially when the draw-off is relatively large) is produced in the division of the waters. The eddies and small whirlpool vortices thus formed, extend to some distance out stream from the structure, stirring the traveling bed load into suspension; whence, much of it is carried into the canal portion and over the head-gate sill, to be dropped again in the more slowly moving canal waters.

When the mixed-up turbulent flow of the river is suddenly quieted by the orderly diversion of, say, 10 000 cu. ft. of water per sec., there is a continued rain of soil to the bottom of the canal. When this is observed for 86 400 sec. each day, the structure responsible for it assumes a significance contrary to its intended purpose. Figuratively, the structure seems to reach down wilfully into the depths of the river to stir up and collect as much of the soil load as possible before it can escape, and place it in the canal on the down-stream side of the head-gate merely to cause annoyance and expense. Then the silt must be transferred back into the river again for final disposal, to prevent its becoming a menace throughout the Canal System.

The writer is of the opinion that a side offtake without under-sluices is an effective selector of bed silt. The drag on the river bed is toward the structure; raising the sill will obstruct the travel of the bed load only temporarily. The relatively small pocket in front of it is filled quickly to form a ramp sloping up to the sill elevation and permitting the travel to continue into the canal. Lowering the sill again causes the entire ramp to move in, offsetting the short respite gained while the pocket was being filled. Such a sill is, therefore, completely ineffective, apart from the turbulence (produced by the division of stream flow), which causes eddies and vortices to lift some of the approaching bed load into suspension to pass it through the diversion gates.

The greatest source of trouble and expense to canal maintenance and operations is removed if the bed load can be excluded entirely, and some added cost to effect this purpose is money well spent beforehand. When "sand is running,"[26] that phenomenon is visible in the disturbed hydraulic gradient and water-surface agitation for a length of several hundred feet in a main canal. The traveling bed sand accumulates in excessive volumes to constrict channelized flow in this canal, the uniform requirements of which are changed to whirlpools and turmoil. Whether or not there is surface evidence of the presence of blocks of sand, expense is involved for bank protection, dredging, or canal enlargement.

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[26] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1113.

The traveling bed load should never pass through the diversion openings above the gate-sill. Furthermore, it is unnecessary to trap it in desilting basins or in dredged sumps. It must not be impeded or retarded in its travel of approach in the river before passing through the structure at an elevation lower than that of the gate-sill. A horizontal separation of the uniformly approaching flow to the structure is needed between the sill and a point some distance up stream, in order that the stream bed shall be kept beyond the influence of the contractural inflow through the diversion openings, thus preventing any disturbance on the approaching bed-load conditions. This can be effected by a light horizontal partition, supported on light walls, the upstream edge preferably having as much overhang as possible. No vortices or eddies can then penetrate to the traveling sand, which is disposed of through under-sluices without any slowing up or trapping by deposition to necessitate rehandling.

A diversion structure, placed across the river normal to the stream, provided with such a partition and under-sluices properly proportioned, will exclude, automatically, all the bed load and that part of the suspended load in the lower water that passes under the partition. Relative to the total suspended load this part is at least in the ratio of the depth below the partition to the full depth of the stream. The logical deduction for such a partition has been described[27] by Mr. F. Y. Elsden, including the final refinement to care for a varied range of water-surface elevations in the river. The refinement will not be necessary at the new Imperial Diversion, however, where a fairly constant river height will be maintained.

A necessary requirement is that the flow of approach be in a smooth uniform channel. When the diversion and under-sluices are passing the full river discharge for long periods, this is not difficult to attain, since the stream tends to stabilize its own approach channel; but when the diversion and open sluices are a part of the river discharge (the surplus passing through floodgates or over an adjacent weir), the stream division, or the accelerated velocities which are produced if the weir is too short, may require a longitudinal training wall extending up stream in the river for some distance, to assure that the approaching bed load of the diversion portion is not disturbed.

The correct weir length that will produce a minimum scouring turbulence with the high discharges, and a minimum of shoaling or deposition with subsequent return to low-water flow, is determinable. Should a long training wall be the solution for uniform approach to the diversion openings, care in determining its location is necessary to avoid shoaling in the approach channel, when the major portion of the river flow is passing through the floodgates or over the weir.

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[27] *Water and Water Engineering*, October 20, 1923.

Figs. 5 and 6 show the diversion works as tentatively proposed, with the changes made to provide the features described herein as being necessary for the successful exclusion of the traveling bed load. The disposal of this soil through the diversion structure is preferably a continuous and unobstructed transportation to a free discharge, which is regulated in volume by the sluice-gates, into the river bed on the down-stream side of the structure. The diversion openings, canal flume, and the under-sluices call for a frontage to the river of 750 ft. and a length of about 450 ft., as shown.

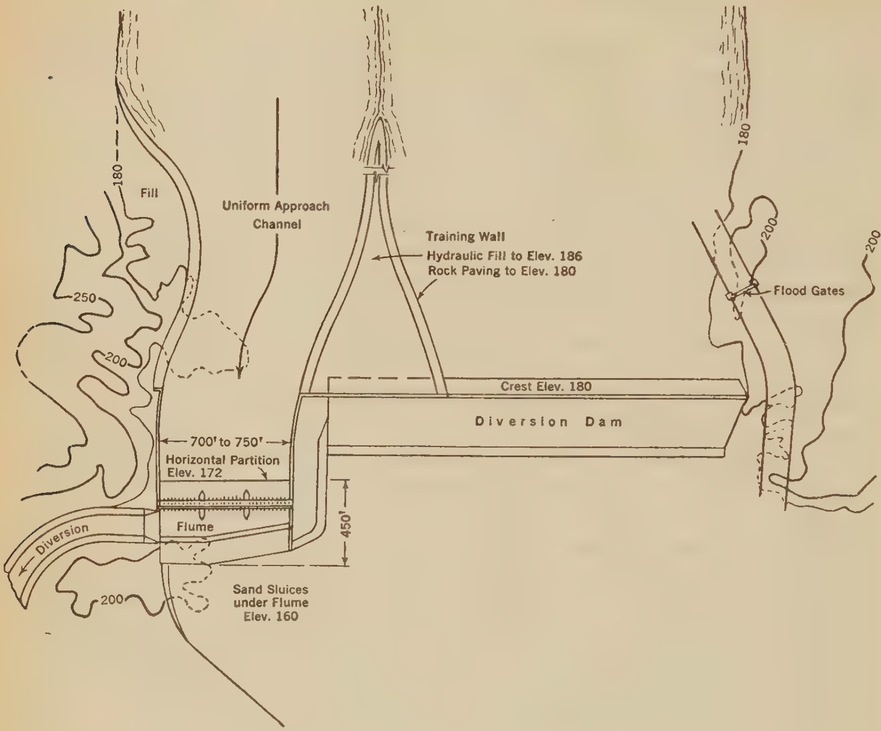
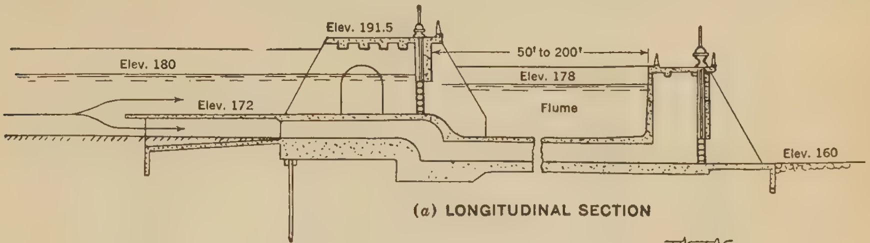

180
Fill
Uniform Approach Channel
250
200
Training Wall
Hydraulic Fill to Elev. 186
Rock Paving to Elev. 180
180
200
200
Flood Gates
Crest Elev. 180
Diversion Dam
700' to 750'
Horizontal Partition Elev. 172
Flume
450'
Diversion
200
Sand Sluices under Flume Elev. 160

FIG. 5.—TENTATIVE PLAN OF DIVERSION WORKS.

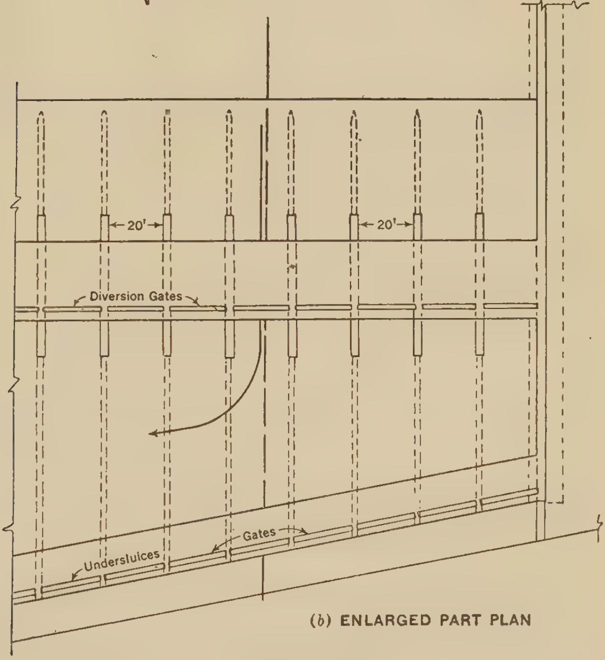
Figs. 7 and 8 show the works if desilting basins are to be used to obtain a partial deposition of the suspended load. The discharge from the under-sluices conveys the bed sand in a conduit that leads across under the lower side of the diversion gates, to empty through a gate-control well, into the river bed as before. Here, the works require a frontage of 1 200 to 1 250 ft. and a length of 1 550 ft. These illustrations are only diagrammatic.

The gates of the under-sluices may be closed to allow the bed load to accumulate. The partial deposition from the lower depth will then raise the bottom of the approach channel to the elevation of the diversion gate-sill, when the gates are re-opened to scour the bed down again. Thus, the under-sluices may be effective for either continuous or for intermittent sluicing operations. A flash-board or raised sill, 1 ft. to 2 ft. high, at the diversion gate would provide further leeway for raising the bed of the approach channel before re-opening the sluices.

A trash rack placed up stream at the under-sluices would prevent submerged driftwood from entering the structure. A footwalk supported on light piers rising from the horizontal partition at the under walls would furnish access for the removal of such driftwood; and flash-board grooves in these piers, with narrow openings through the horizontal partition to permit the lowering of the flash-boards, would be a means of closing the under-sluices in case of an emergency, or for inspection.



(*a*) LONGITUDINAL SECTION



(*b*) ENLARGED PART PLAN

FIG. 6.—ENLARGED PLAN AND LONGITUDINAL SECTION OF DIVERSION WORKS.

Mr. A. B. Buckley states[28] that the direction in which the gates travel when opening, is an important feature for the exclusion of bed sand. Referring evidently to offtakes without under-sluices, he announced in 1922 that the difficulty due to silt accumulations at the heads of canals in India had been quite overcome. One canal was dredged every year for twenty years until it was discovered that if instead of the vents being opened from the bottom, they were opened from the top, the bed of the river was not scoured into the canal. This was said to be an almost infallible cure, but might not always be possible financially. Mr. Elsden has reported[29] that silt has been a source of serious trouble on many canals in India, but on none of them has it been necessary to resort to constant dredging.

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[28] *Minutes of Proceedings,* Inst. C. E., Vol. CCXVI, pp. 215–216.

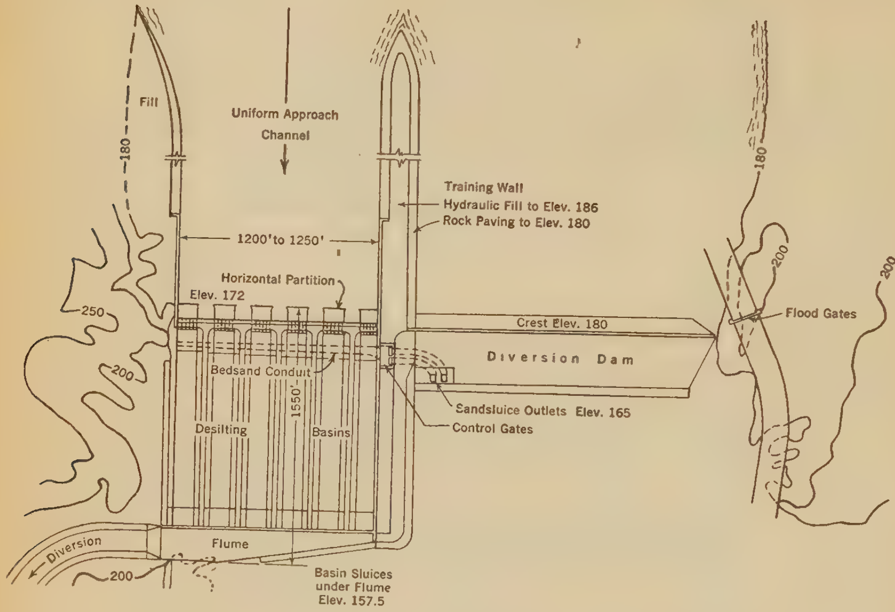
Fill
180
Uniform Approach Channel
Training Wall
Hydraulic Fill to Elev. 186
Rock Paving to Elev. 180
180
1200' to 1250'
Horizontal Partition
Elev. 172
250
200
200
200
Flood Gates
Crest Elev. 180
Diversion Dam
Bedsand Conduit
1550'
Desilting
Basins
Sandsluice Outlets Elev. 165
Control Gates
Flume
Diversion
200
Basin Sluices under Flume Elev. 157.5

FIG. 7.—TENTATIVE PLAN OF DIVERSION WORKS WITH DESILTING BASINS.

DESIRABILITY OF INSTALLING DESILTING BASINS

The bed sand and heavier silt particles are quickly precipitated if the flow is stilled or greatly retarded; but this is not so for the suspended load, which requires a period of detention in quiescent water for several hours.

The rate of deposition of soil in suspension in still water from the Nile River in Egypt, was found[30] to be 2 min. through a depth of 25 cm. for the sands which comprised 10% of the sediment, and 3 days for the silts and clays to settle through 50 cm. The rate for the Colorado River was found[31] to be 2 hours through a depth of 1 m., to the extent that a definite mud deposit is formed, leaving the water apparently clear to the eye if not closely inspected.

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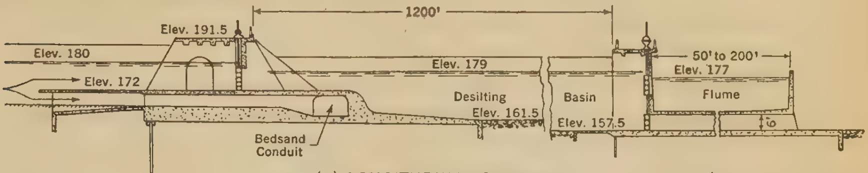
[29] *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI, pp. 215–216, 225.

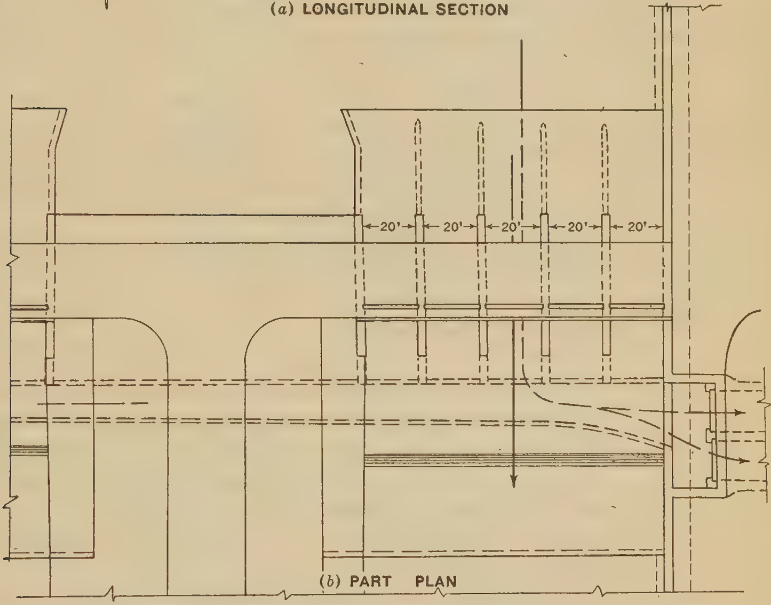
[30] *Loc. cit.*, Vol. CCXIX, p. 115.

[31] Seventh Biennial Rept., Dept. of Eng., State of California, p. 119.

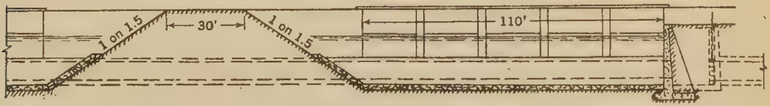
If the flow in a long section of canal prism is stilled for sufficient time, the fine suspended silts settle slowly until they remain very lightly intersupported as a loose sludge on the bed. The slightest movement of the water, such as the resumption of flow, causes the settlement to be disturbed and the silts, which are readily susceptible to diffusion in fluid motion, are again in suspension unless precautions are adopted to minimize the disturbance of this undeposited sludge. One such precaution would be to have the stilling basin relatively wide at right angles to the canal flow, drawing off the quieted surface water from a point on the wide side. The inflow to the basin should be directed toward those wide parts most remote from the position of the outflow.


1200'
Elev. 191.5
Elev. 180
Elev. 172
Elev. 179
Desilting
Elev. 161.5
Basin
Elev. 157.5
50' to 200'
Elev. 177
Flume
10
Bedsand
Conduit

(a) LONGITUDINAL SECTION


20'
20'
20'
20'
20'

(b) PART PLAN


1 on 1.5
30'
1 on 1.5
110'

(c) CROSS SECTION

FIG. 8.—ENLARGED PLAN AND LONGITUDINAL SECTION OF DIVERSION WORKS WITH DESILTING BASINS.

Each 1 000 sec-ft. of flow stilled for 1 hour would require a basin capacity of 80 acre-ft., which is equivalent to a water area of 4 acres with a vertical depth of 20 ft. If the maximum diversion of 15 000 sec-ft. is to be detained for 1 hour, a capacity of 1 200 acre-ft. (or fifteen such basins), is needed, with as many more providing the draw-off to maintain constant diversion flow. Thus, thirty such basins with a water area of 120 acres (requiring a total area of, say, 200 acres of ground) are necessary to obtain only 1 hour's detention of the flow, not even considering desludging operations and time required to fill the basins. A further increase of probably 50% would be required for the desludging operations. Each additional hour of detention would be a direct multiplication of the extent given.

The tentative layout in Fig. 7 shows six basins on an area of about 37 acres which is equivalent to twelve city blocks, each 300 ft. square, inclusive of streets 60 ft. wide. It could be effective only for the removal of the bed sand and coarse silts, by effecting a retardation only, of the velocity of flow. Even for this limited area a cost item of construction for desilting works is stated to be $3 375 000.

Are these great costly basins necessary? The bed load can be eliminated without the intervention of desilting basins, but the suspended loads cannot be so eliminated and then decreased only in a slight degree, since the necessary desilting area and desludging operations assume too great a magnitude to permit of sufficient detention of large diversion flows.

It has been reasoned in this paper that the suspended silts diminish in quantity as the changing river gradients gradually become stabilized to the new conditions below the Hoover Reservoir, and that after some years the clear-water flow will transport only bed sand.

New irrigation laterals and ditches, required on 300 000 additional acres to be irrigated in Imperial and Riverside Counties, California, will be constructed in sandy soils which, unlike the present irrigated area of Imperial Valley, are not composed of Colorado River alluvium. It may be a double mistake to deprive these new watercourses in porous soils, of the beneficial effect of silty flow, and to deprive the ranchers who are reclaiming windswept sandy acreage from the silt that would be added to his fields if the desilting basins were effective in its removal.

Water containing silt would be an assistance proffered by Nature to these new developments, and advantage of it should be taken before the laterals begin to carry the subsequent clear-river water. Silty flow is advantageous because: (1) It contributes to the prevention of seepage, the fine silt particles gradually filling the interstices in the loose sandy canal sides and bottom, thus sealing these seepage passages; (2) it helps to maintain depth by giving stability to the side slopes of canals in sandy soils; (3) it will kill all aquatic growth by excluding light; and (4) it will give added surface soil which is also claimed to have a fertilizing value, to the rancher who has sandy land.

However, this added soil is of negative value to the long-suffering ranchers who farm the heavy alluvium of the present irrigated area; and impatience to obtain clear water is a factor that must be considered, if such water can be obtained even at high cost. A sudden conversion to clear-water flow will undoubtedly result in additional costs for canal maintenance, and this is an adverse factor to the change.

The arguments in this paper can be summarized as follows:

(*a*) The traveling bed load can be removed without desilting basins.

(*b*) The traveling suspended load cannot be greatly reduced with desilting basins, because the period of detention must necessarily be too long for precipitation of the fine silts.

(*c*) The suspended load will be a diminishing quantity with the stabilizing of new river gradients, and will not exist after a few years.

(*d*) Silty canal flow is desirable to new acreage during the years after the porous soils comprising the mesa lands are first reclaimed.

(*e*) Impatience and sentiment from the large agricultural communities of Imperial Valley may demand that all possible efforts be made to obtain clear-water flow in the canals, when the new diversion is effective.

Items (*a*), (*b*), (*c*), and (*d*) do not favor justification for 40 to 100 acres of huge desilting basins with large desludging structures, and subsequent daily sluicing operations. Item (*e*) may overrule the other four, in which case competent investigation, with some research, may be well worth while to determine factors governing the rapid desilting of fine silts, in order to obtain a minimum magnitude of works and of subsequent operations.

References have been made in Item (9) and in other places as to the colloidal phenomena in Colorado River water.[32] Particles of colloidal dimensions cannot be precipitated without the presence of an electrolyte, which may have to be added to cause flocculation. However, it is not believed that the term, "desilting," is intended to apply to this requirement for the purposes of clarifying irrigation water.

CONCLUSION

The assumptions used in this paper to estimate volumes are necessary to visualize the greatness of the transported loads. They are very conservative; even greater values for the basic dimensions would be nearer to actual conditions. If cross-sections could be taken simultaneously 10 miles apart over a 100-mile stretch of the Lower Colorado River, weekly between March 15 and the day of the flood peak in late May or June, actual data would be available for interesting comparisons.

If a suggestion has been made pointing to a less cumbersome solution to the problem of soil exclusion at the new diversion than that of constructing the necessarily enormous desilting basins, the effort taken to prepare the paper will have given some satisfaction. A desilting structure on the lines suggested, will be as efficient as the basins; it will be more economical in first cost of construction and in operation; and it will be far less unwieldly because flow velocities are not interrupted. With such a structure available when the river flow has become clear, there will be no reason for the basins, but without it they will always be necessary.

The conclusions are briefly stated as follows:

(1) The volumes of soil transported by the Colorado River have been greatly misconceived, chiefly due to the generally accepted viewpoint that

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[32] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1130.

the bed load of flood discharges is stirred up to constitute the suspended load. This is disproved by the greatness of the cubic contents moved by recurring deep scouring during short periods, at least once each year and almost without exception.

(2) The bed load is the principal source of trouble; it contains a much smaller percentage of the fine silts than the suspended load. The difference in the shapes of soil particles offers an explanation for the chief characteristics that distinguish between the bed load and the suspended load, respectively. Granular shapes, when raised into suspension by vertical eddies, tend to sink when they are released from the vertical influence. Therefore, in transportation, they are most dense close to, and along, the bed. On the other hand, the laminar shapes, which in quiescent water sink with their flat sides approximately horizontal, are susceptible to diffusion and retention with the least movement of the fluid in which they are supported, so that flow conditions find the suspended silts uniformly distributed throughout the stream section, for the entire length of the river or canal.

(3) The exclusion of the bed load from any large diversion flow is more important and can be accomplished more easily than that of the suspended load.

(4) Desilting basins of minimum size will trap the bed load by retarding the velocity of flow, so that it can be disposed by subsequent sluicing operations. However, they will not desilt the suspended loads unless it is possible to stop the flow completely for a number of hours. This requirement calls for desilting areas of such magnitude as to be cumbersome and economically non-feasible.

(5) Silty canal flow, without a traveling bed load, has several advantages to the reclamation of 300 000 acres of undeveloped lands on porous soils.

(6) Turbid conditions will gradually lessen and practically cease to exist after an unknown period of several years, when the changed river gradients have become established below the Hoover Reservoir, due to the clear-water discharge from it.

(7) Bed sand will always be in transportation, even when clear-water flow has been established, the silts being first washed from the alluvial channel bed, by meandering of the stream and recession of gradients. With flood velocities modified, the transportation of sand will be slower but continuous, due to additional supplies being contributed yearly from tributary streams and sand washes. This is also due to the fact that the bed becomes depleted of the cohesive silts, and the stream area will then acquire a wide and shallow section, and relatively a greater bed surface for transportation; the velocities, even with minimum discharges, will always be sufficient for moving sand grains.

(8) It is possible to build a diversion structure, with stream approach control, such that it will exclude the traveling bed load automatically, and this would eliminate the necessity for cumbersome sand-trapping basins (the word, "desilting," is not truly applicable) and desludging operations. Such a structure will save money in the first cost of construction and in the costs of operation and maintenance.

(9) Construction of the new head-gate will probably not proceed before the Hoover Dam is advanced sufficiently toward completion to provide effective control of up-stream floods. In the meantime, competent investigation and research on problems pertaining to the silts and sands in transportation, especially within the range of those discharges expected in the future, and on the efficiencies of desilting that can be obtained, will eliminate much of the present guesswork and will enable development of the works to proceed on sound lines, from established findings. Certainly, this is a justifiable aim in view of the permanence and importance of the control structure.

ACKNOWLEDGMENT

Material for the preparation of Figs. 5, 6, 7, and 8 of this paper, was made available through the courtesy of Raymond F. Walter, M. Am. Soc. C. E.

DISCUSSION

C. E. Grunsky,[33] Past-President, Am. Soc. C. E. (by letter).—It is with satisfaction that the writer notes a reviving interest in some of the larger problems of the Colorado River. Of this reviving interest, this paper on a method of freeing the river water of a large proportion of its bed load of silt at the head-works of the proposed All-American Canal, gives evidence. The subject is certainly one which merits the careful attention and study of the Engineering Profession. The magnitude of the problem has gone unquestioned and yet, to the present time (1933), how little is known of the physical characteristics of the silt particles, carried by the waters of the Colorado, which are to be handled! What are their dimensions? What shapes predominate? How much colloidal matter is with the silt? It is indeed refreshing to have the author emphasize the magnitude of the bed load of the Colorado. In this matter, however, the writer is not in full agreement with Mr. Rothery. He is willing to concede, and in an earlier paper[34] has endeavored to demonstrate, a large bed load, but found nothing to warrant the author's intimated conclusion (see heading "The Bed Load") that the bed load of the river exceeds its suspended load.

The writer is in full accord, too, with the author's contention that there will be silt problems at points of water diversion below the completed Hoover Dam, even if in the course of time these may be greatly simplified. The writer is, for example, still among those who do not understand the reason why the Metropolitan District did not take advantage of the desilted supply of clear water which will be available at the Hoover Dam, instead of selecting a diversion point for domestic water far below it, where the engineer will have to contend with all the annoyances of a turbid river supply.

The writer does not care to enter upon a discussion of the merits of the author's plan for ridding the water of its bed load at the head of the proposed All-American Canal, but in this connection, would merely say that if the bed load can be eliminated, the problem of canal operation and maintenance becomes relatively simple. This is due to the fact, well established by all the available observations, that Colorado River water, entering a canal from the river, will carry its suspended load practically without change throughout the length of the canal. Any device for removing or, at any rate, for greatly decreasing the bed load, should be simple and relatively inexpensive. The nearer it can be kept in design to a sedimentation basin with suitable sluicing facilities the better.

Ivan E. Houk,[35] M. Am. Soc. C. E. (by letter).—The author has contributed a comprehensive, timely, and interesting review of the many important problems involved in the measurement and control of silt transportation in the Lower Colorado River.

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[33] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif. Mr. Grunsky died on June 9, 1934.

[34] "Silt Transportation by Sacramento and Colorado Rivers and by the Imperial Canal," by the late C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1124 *et seq.*

[35] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

Many of the data used as a basis for his discussions are of an approximate nature. Consequently, his conclusions can only be considered approximate. Probably the greatest value of his review is the emphasizing of the complexity of the silt problem and of its importance in future river-control work.

It is indeed unfortunate that accurate data are not available to show the average depth of periodic flood scour in the section of the Colorado River under discussion. Isolated observations of scour may be very misleading as regards representative conditions in a 300-mile stretch. River gauging stations are usually located at cross-sections where the stream is more or less contracted in width. Therefore, depths of scour observed at such stations usually tend to be greater than the average rather than smaller. Any computation of bed load based on an estimated average depth of scour is nothing more or less than a pure guess. It may be several hundred per cent. in error in either direction.

The writer believes that the author's guess of a spring flood bed load equal to twice the estimated annual suspended load is much too large, instead of being conservative as the author concludes. It is understood that river-bed excavations at Hoover Dam show that the depth of periodic scour in Black Canyon has been less than was formerly supposed. Actual measurements at other locations might show similar conditions. In order to make a fairly accurate estimate of the average depth of periodic flood scour in the river channel above Yuma, Ariz., it would be desirable to have detailed cross-sectional measurements during flood periods at locations about ten miles apart along the entire length considered, as suggested by Mr. Rothery. Even if satisfactory information were available regarding the average depth of scour, it would not be possible to make an accurate calculation of the total bed load, since a large proportion of the scoured material undoubtedly is carried in suspension. The estimate of total bed load based on an average depth of scour would necessarily have to make proper allowances for the suspension factor.

In rivers flowing through gravel and boulder formations undoubtedly there is a definite boundary between suspended and bed load silt. Such a condition may exist, temporarily, in some sections of the Lower Colorado River, at times when the erosion is deep enough to reach coarse gravel or boulder deposits. However, it is probable that during the whole, or, at least, during the greater part of the Lower Colorado River floods, there is a gradually increasing thickness of the mixture of solid particles and water near the bed of the river, rather than a definite line of demarcation. This is indicated by the difficulty in finding a solid bottom while making soundings during flood periods. The writer has experienced such difficulties in gauging floods in rivers flowing through much coarser materials than those characteristic of the Lower Colorado.

In measuring the silt content of the Colorado River flow at Yuma, approximately one-third of the samples were taken as close as practicable to the bottom of the river. Consequently, it is believed that the estimated silt flow based on the Yuma measurements includes a large proportion of the material eroded along the bed of the river at up-stream locations, material considered by the author as making up an unmeasured bed load flow.

The accurate measurement of silt flow is always a difficult matter, whether the material is moving in suspension, by saltation, or as a bed load. The writer cannot agree that where samples are taken near the bottom of the river, with the sampling bottles pointed up stream, the "increased percentages are properly part of the unknown bed load." Holding the bottles in such a position naturally results in catching more solid particles than are actually present in a given volume of flow. Such results cannot properly be used in calculating the silt load of the stream on the basis of a current-meter gauging of discharge. In order to determine the correct proportion of silt present in river flow it is necessary to adopt some method of segregating actual samples of the flow instantaneously at different locations in the cross-section, such as a long cylinder, open at both ends, placed in a direction parallel to the lines of movement and arranged so that the two ends of the cylinder, or the two ends of a central section thereof, can be closed simultaneously. A silt sampler was developed to operate in this manner.[36]

The author is undoubtedly right in his conclusion that "both suspended and bed loads will be present in the curtailed river flows below the Hoover Reservoir." However, his expectation that the suspended loads will "decrease in extent to almost negligible quantities as the regimen of the stream becomes stabilized" may or may not be fulfilled. As soon as Hoover Dam is put into operation, cloudburst occurrence, resulting cloudburst erosion, and details of surface geology on the tributary drainage areas below the dam will become important factors as regards the character and quantity of solid materials transported by the regulated flow. The writer is not familiar with the surface geology of the drainage areas of the Lower Colorado River. If there are large areas of extremely fine surface silt deposits in the lower tributary areas, and if cloudburst rainfalls happen to fall on such deposits, large quantities of material will certainly be eroded and washed into the main channel and will tend to keep up the turbidity and supply of suspended material.

The construction and putting into operation of the Elephant Butte Reservoir in New Mexico changed the soil transportation problem of the Rio Grande at El Paso, Tex., from what was essentially a silt problem to what is now essentially a sand problem, due to the fact that the tributary drainage areas below the dam are covered with sand and gravel deposits rather than with fine silt or clay deposits. The change would have been quite different if the Rio Puerco, of New Mexico, had joined the Rio Grande below Elephant Butte Dam, instead of above it. The Rio Puerco flows through deep and extensive deposits of fine silt. Its flow contains large quantities of silt during normal stages as well as during flood periods. A detailed study of surface geology, cloudburst occurrence, and cloudburst erosion on the tributary drainage areas of Colorado River below Hoover Dam would undoubtedly aid in formulating accurate predictions regarding the character and extent of the silt load to be expected after the reservoir is put into operation.

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[36] "Silt in the Colorado River and Its Relation to Irrigation", by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., *Technical Bulletin 67*, U. S. Dept. of Agriculture, p. 12.

H. M. Rouse,[37] Assoc. M. Am. Soc. C. E. (by letter).—An opportune and valuable study has been made by the author, indicating the need of drastic changes in the original published plans of the desilting basins for the proposed new Imperial Valley Diversion Works. He emphasizes particularly the large quantity of bed sand present in the river flow, which can best be handled by methods that differ from those used in handling the silt or suspended load.

The bed sand should be diverted neither into the Main Canal nor into desilting basins unless it may be proved by various experimental models of the diverson works that it cannot be separated and caused to flow continuously back into the river. Mr. Rothery's suggested plan for this part of the diversion works is a fine basis for beginning experiments.

The writer realizes from experience on the Imperial Valley Canal System, that bed sand is the most troublesome and costly factor in canal operation and maintenance, but believes that the suspended load, or silt, should also be removed, and that there will be enough of it in the future to warrant the construction of desilting basins. Although ranchers of the present irrigated area do not agree as to the value of the silt on the lands, a large majority believe it has little or no value as a soil renewer, and that it is a detriment to many crops.

The removal of bed sand will decrease greatly the cost of lateral canal cleaning in the present irrigated area. Silt removal will further decrease that cost and will stop the growth in the size of the unsightly banks or levees of the canals caused by cleaning out the silt and sand deposits. Bed-sand removal will allow of the installation and successful operation of underground pipe-line water distribution systems on the ranches, which are not now feasible on account of lack of slope or opportunity for sluicing the pipe lines. Silt removal will allow the use of pipe lines on very light slopes, or on whatever slopes clear water requires.

Possibly bed-sand removal will not require additional structures for slope control in the existing canal system, while silt removal will probably require some construction for slope control or against bank erosion, or both. However, this construction will come gradually and, in some cases, will permit of improvements in operating conditions. Furthermore, the slope control may be obtained in many cases at the time of building more diversion structures, which will be required from time to time as the lands are subdivided into smaller units.

The writer believes that the desilting basins are needed, in addition to the desanding works. However, the location for the desilting basins can be chosen where the large areas (which the author rightfully emphasizes are needed) are available without expensive quarrying in the mountains or expensive foundations in the river bed. This points to a location below the proposed diversion works where the Main Canal location begins to swing away from the river; or in the vicinity of Araz Junction, Calif., where the location is again near the river; or between the latter and Pilot Knob, Calif., where the location of the Main Canal can be near enough to the river to afford ample slope for sluicing the basins.

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[37] Engr., Colorado River Land Co., S. A., Calexico, Calif.

A main canal slope can be used which without altering the basic project scheme, will carry the silt to the desilting basins, and the basins may be designed, constructed, and operated, so as to leave any percentage of silt desired in the irrigation water for the stabilization of canal banks in the sandy mesa lands and for the enrichment of the lands themselves. Should it be decided to leave some silt in the irrigation water for the last-mentioned purposes, additional desilting basins can be built at the western edge of the mesa lands, to clear the water for use in the present irrigated area. The construction of the desilting basins away from the diversion works would allow more freedom in the design of the desanding works.

The construction and testing of models of the diversion works, with desanding and desilting devices, and of models of desilting basins, if not already done, are needed for the final design of the head-works. Model study is needed, especially for the following (referring to Figs. 5 to 8): (*a*) Determination of the minimum necessary distance from the up-stream edge of the horizontal partition, to the irrigation water-control gates; (*b*) thickness of the lip of the horizontal partition and the material to be used in its construction; (*c*) shape and length of the approach channel; (*d*) type and location of drift-handling booms, gratings, or machinery; and (*e*) avoidance of the formation of bars at the mouth of, or in, the approach channel with floods of varying stages and duration, since it is possible that provision of more flood-gates in the dam proper, with less resulting variation of stage during floods, may be found desirable.

The enemy of desanding or desilting, of course, is turbulence. The author refers to "a side offtake without under-sluices" as being an "effective selector of bed silt." The head-gate of one of the main canals of the present canal system is a particularly good example of this. Although supplied with raised flash-board "sill" or overpour control, the water diverted carries its proportion of the bed sands traveling in the Main Canal above the diversion.[38]

At a point in another of the main canals a structure was built which combined the purposes of a check or control structure in the Main Canal, a head-gate for another canal, and an important waste-gate. The placing of the waste-gates some distance up stream from the other gates and in the bed of the Main Canal, was seriously considered, with the idea of removing a large amount of the traveling bed sands with the water wasted at this point. Practical difficulties prevented such location, and the waste-gates were placed vertically below the gates of the irrigation water control, the waste water entering a tunnel, the roof of which is the floor of the irrigation water-control gates above, the tunnel emptying at the side of the combined structure. Turbulence in front of the structure causes the bed sands to be carried on with the irrigation water as was expected, the percentage of sand in the irrigation water and the waste water being practically the same. This is a case of take-off in a direct line with the Main Canal, and with under-sluices provided, although, of course, the under-sluices could only be used to the extent that water was wasted.

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[38] Reference is made to Photograph B, Pl. 2, *Technical Bulletin No.* 67, entitled "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., which shows the bed of the aforementioned canal below the head-gate.

One of the troublesome effects of bed sand in the main canals of the present irrigation system is the fluctuation of the discharge at one point, with no changes at the next control point above. This trouble is evidenced by increases and decreases of hundreds of second-feet in the discharge at key waste-gates, the change amounting to a maximum of 700 sec-ft of a main canal flow of, say, 5 000 sec-ft. For some time gauges were read on a stretch of the Main Canal, and these readings indicated a raising of the water surface, because of the bed sands being temporarily deposited; then they indicated a lowering of the water surface, releasing the water which had been temporarily "stored" by the building of the bed-sand "dam" in the bottom of the channel. In some cases, a cycle of one of these fluctuations would take place in 5 or 6 hours and, in other cases, the decrease might be slow and the increase very sudden, the cycle taking 24 hours, or more. Sometimes the increase only would be noted, the decrease presumably being hidden in daily water handling, or in a very slow decrease of some days' duration. In order to cover the great fluctuations caused by the bed sands, large waste-gate capacity and a great quantity of water are required in order to avoid fluctuations of delivery to main canals, laterals, and, thus, to irrigators.

Seasonal movement of bed sands in the Main Canal also causes trouble in the spring, the bed of the canal having been built up during the low period of flow in the winter. In the past, lack of free-board or slope (or both) has restricted free handling of the water, requiring that increases in the Main Canal discharge be made in small quantities to give the bed sands opportunity to move out and thus increase the water area of the canal. Similar bed-sand phenomena on the Colorado River are generally known; but the necessity is not generally known for the Imperial System to carry large waste-water reserves to cover large fluctuations, the sources of which are on the main canals themselves.

William T. Collings, Jr.,[39] M. Am. Soc. C. E. (by letter).—Interesting data on the problem of soil transportation have been presented in this paper. The author has provided a conception of the magnitude of the enormous quantity of material carried by the Colorado River, both in suspension and along its bed. This problem is one that has confronted engineers on irrigation projects in the Lower Colorado delta region for years, and much has been written on the subject. As the author suggests, a thorough understanding of these data is necessary by those engaged in the design of irrigation works on, or leading from, the river.

The writer is particularly interested in a consideration of the merits of a plan proposed by the author for eliminating the desilting basins shown on the

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[39] Engr., Imperial Irrig. Dist., Imperial, Calif.

Government design by substituting a horizontal partition separating the flow to the head-gate structure. The author makes the following statement:

> "A diversion structure placed across the river normal to the stream, provided with such a partition and under-sluices, properly proportioned, will exclude, automatically, all the bed load and that part of the suspended load in the lower water that passes under the partition."

A number of questions arise as to whether this statement can be borne out in entirety in application to the head-works of the All-American Canal, considering factors of major importance, such as magnitude of the structure, fluctuating river flow, available sluicing heads over and above irrigation requirements on the canal at periods of low-river discharges, shoaling, dependability of operation, maintenance, etc.

Under normal or average conditions of river inflow to Boulder Canyon Reservoir immediately after the completion of Imperial Dam—and for the first few years of the operation of the head-works, and when power demands both on Hoover Dam and on the Canal System have not been fully developed—there should be no occasion for close conservation of sluicing water at the head-works. However, firm power from Hoover Dam has been based upon output obtainable in low run-off periods as determined by run-off records from the Colorado River over a period of thirty years or more. The same conservative principle should be applied in the design of desilting works as regards capacity for sluicing operations in anticipation of possible low-period discharges. The possible demand for diversion at Imperial Dam as of date of completion will be close to 12 000 sec-ft, taking into consideration water to the Yuma Project, present Imperial and Coachella irrigation demands, and power installation. Assuming a maximum of 3 000 sec-ft, or the normal head-gate capacity of one of the desilting basins as sluicing head, the river flow required at the head-works would be 15 000 sec-ft. Under full development of the canal project, the full capacity of 15 000 sec-ft for the first reach of the canal to the drop for the Yuma Project is anticipated. Sluicing head then becomes a quantity over and above 15 000 sec-ft. It can be seen that under conditions to be anticipated during periods of low-river flow, approaching those of firm power output at Hoover Dam, economy in sluicing head is a major consideration in design.

The multiple-basin plan seems to offer a more dependable and conservative design, from the standpoint of sluicing head, than that offered by the author. While no definite quantity has yet been determined as to the minimum required under the full capacity of diversion with the six basins in operation, the writer believes that 3 000 sec-ft would be quite adequate, if not much in excess of requirements under the heaviest silt content, in so far as cleaning of the basins is concerned. No doubt, larger heads will be necessary periodically to clear the river channel below the structure. Referring to Fig. 5 of the tentative plan shown by the author, using the width of 700 ft in the uniform approach channel and 8 ft as the depth of water in that part of the water cross-section above the horizontal division wall (or what might be called a "water splitter"), the resulting mean velocity of approach for

15 000 sec-ft of water becomes 2.68 ft per sec. Assuming 4 ft as a minimum depth of water below this wall, the water cross-section below the wall line produced and immediately up stream from the inlet to the under-sluices, is 2 800 sq ft. The mean velocity of approach in the full cross-section would be approximately at the location of the horizontal division (or about 0.7 ft above). The mean velocity in the lower part of the water section would depend somewhat upon the friction factor on the bottom of the approach channel, 1.7 ft being a safe minimum value to use. The mean velocity in the full water section would be approximately 2.4 ft per sec. Assuming that these velocities remain unchanged at the entrance to the under-sluices the discharge through the sluice-gates would be 4 760 sec-ft (neglecting the cross-section area of the longitudinal supporting walls and horizontal partition).

Considering inlet-gate capacity to the canal flume alone, the width of 700 ft, or 750 ft, as shown on the tentative plan, does not utilize all the 2.0 ft of head available. Using 0.6 ft as the approximate loss of head through trash racks, the net velocity head would be 1.4 ft, resulting in an intake velocity of about 5.75 ft per sec. Using 7.5 × 20 = 150 sq ft, as gate-opening, the capacity of one gate is 860 sec-ft, and the total width of the gate structure would be more nearly 400 to 450 ft. The increased velocities resulting in a narrowed approach channel would be reflected in the lower section of the water cross-section. To compare the sluicing head, consider a 400-ft width of channel, of the same water depth and like dimensions, 8 ft above the partition line produced up stream and 4 ft below this line, the respective areas in section would be 3 200 and 1 600 sq ft. The mean velocities in the upper and lower sections would be 4.68 ft and in excess of 3.0 ft per sec, respectively. The resulting mean velocity in the full water cross-section would be about 4.2 ft per sec. Assuming, again, that these velocities were unchanged at the entrance to the under-sluices, the quantity of water passing under the partition and through the sluice-gates would be, conservatively, between 4 800 and 5 000 sec-ft. This comparison is not a suggestion that the width of the gate structure and approach channel in such a design should be limited to the minimum required for the maximum diversion quantity. Provision for fluctuation in pond depth above the structure would be necessary to provide against overtopping the overflow weir at low discharges, which is accomplished by a widened structure. The writer believes that the multiple-basin plan would prove to be more economical in sluicing water by an amount of between 2 000 and 3 000 cu ft per sec, under normal or low discharges.

There will be long periods when the diversion and under-sluices would be passing the full river discharge, and during such periods, if likewise long periods of unvaried inflow to the canal were maintained, such a design as the author suggests might function satisfactorily in segregating the bed load of soil. There will necessarily be fluctuations of heads to varying demands from irrigation and power, and these fluctuations must be controlled by co-ordination of operation of the intake gates to the flume and the under-sluice gates at the ends of the sluice culverts. Under most favorable conditions, in order to maintain undisturbed velocities at the intakes it would seem that each battery of gates for the full width of the structure would have to be operated as a unit. Any material variance in discharge through adjacent sluice-ways would result in a disturbance in approach velocities creating turbulence sufficient to put large quantities of the heavier materials into suspension and carried above the partition and into the flume. As a matter of practice it would seem that such a co-ordination of gate operation, with about fifty gates involved, would be most questionable of attainment.

Considering the effect of longitudinal supporting walls and the installation of trash racks at the entrance to the sluice-ways, each of such walls becomes virtually a "water splitter" in a vertical plane. If the horizontal wall could be permitted a considerable over-hang up stream, these might have but little effect upon the approaching water velocities. No over-hang of the horizontal wall would be permitted up stream from or above the trash racks. Therefore, any over-hang must necessarily support the racks from lateral pressure. The cantilever action in the over-hang would limit its length to that of a short lip unless heavy beam reinforcement was added, which beam construction would be objectionable as an entrance factor. Submerged débris lodging against the trash racks above the under-sluices would, at best, be difficult to remove; but if lodged under any permitted over-hang it would be almost impossible to remove under flow conditions. It is thought that the combined effect of trash racks and vertical partition walls would be adverse to a continuous and unobstructed transportation to a free discharge of the bed soil.

Another serious consideration in the design of a structure of such proportions, particularly with a large number of gates, is the matter of maintenance. Accessibility to gates and sluice-ways in cases of repairs is necessary, and this with the minimum interruption of diversion and sluicing operations. The maintenance of the basin structure would be comparatively simple where it is possible to cut out one or more basins or sluice-ways without seriously impairing or interrupting the operation of the others.

The aggregate capacity of the tentative design for the dam and head-works by the Bureau of Reclamation is 259 000 sec-ft. It is unlikely that under regulated river flow the head-works will ever be called upon to utilize any quantity closely approaching that capacity. However, it is not unlikely that the river discharge at Imperial Dam will exceed, perhaps materially, 110 000 sec-ft prior to provision for other storage works below the Big Williams water-shed. From experience by the Imperial Irrigation District, the Yuma Project, and other agencies in dealing with the control of the Colorado River under flood discharges, it is hazardous to predict what may be the extent of shoaling and channelizing above any structure that tends to divide the flow of the river. It seems reasonable to believe that, under a sudden increase in the flow of the river from a comparatively low discharge to about 100 000 sec-ft, with a quickly receding river thereafter, that at times sufficient material would be deposited at the head of a trained approach channel to the diversion works to require removal before normal flow conditions could be restored, or at least to interrupt inflow to the diversion gates. In looking forward to the operation and maintenance of the head-works, naturally, the engineers of the Imperial Irrigation District are hopeful that the design eventually adopted will preclude any possibility of dredging operations.

At the time the location surveys were being made by the Bureau of Reclamation for the All-American Canal, the engineers of the Imperial District did some research work on certain of the main canals in determining hydraulic elements of flow. Of particular import was the adoption, as nearly as could be determined, of the proper value of the coefficient of n to be used in the design of canal sections. The canals studied were the Alamo Canal (the main canal through Mexico), at Alamo Mocho Station; the East Highline Canal at "B" Heading; the Central Main, Briar, and West Side Main Canals at the International Boundary Line; and the West Side Main Canal at what is known as the Drain Station.

In Table 1, the values of A, V, and Q were obtained from current-meter records of discharge and the values of p and r were computed. The slopes of the canals were determined by field measurements and C was computed from the Chezy formula: $V = C \sqrt{r s}$. The values of n were determined from a diagram of functions computed from Kutter's formula. Considerable thought was given in these studies to the determination of non-silting and non-scouring velocities.

At Alamo Mocho Station, the Alamo Canal is known to silt and scour more or less. No doubt, this explains the erratic values in Table 1 (*a*), which shows a very low value of n at medium discharges and high values at high and low discharges, when silting and scouring, respectively, occur.

Referring to Table 1 (*b*), it is to be noted that the results show a very consistent and relative increase of velocity, with the hydraulic radius. They also show a quite consistent value of the friction coefficient, say, a mean of 0.0185. In Table 1 (*c*), the variable n-values for flows between 100 and 400 ft per sec, are probably due to the effect of silting at low velocities. The mean value of n in Table 1 is approximately 0.020.

The West Side Main Canal just below the meter station at the International Boundary should not be considered typical of a non-silting and non-scouring canal. The sides and bottom of this canal show a hard and tight clay, and an eroded rough surface. This results in the unusually high coefficient of friction as evidenced by Table 1 (*d*).

At the Drain Meter Station, the West Side Main Canal is practically self-maintaining. What silting may take place at low heads is relieved when these heads increase, and no cutting of the banks takes place at maximum heads. It is to be noted in Table 1 (*c*) that the functions are quite consistently related through a range of flow from 450 to 650 cu ft per sec. The variation of Kutter's n is also uniform.

The Briar Canal functions under a light grade and comparatively slow velocities, which tend to permit the rapid deposition of silt under normal heads. The Briar Sluice at Birch Canal was installed after the meterings

TABLE 1.—Studies to Determine Hydraulic Elements of Flow

Date	Area, A, in square feet	Wetted perimeter, p, in feet	Hydraulic radius, r	Velocity V, in feet per second	Flow, Q, in cubic feet per second	Chezy coefficient, C	Kutter's n
(*a*) Alamo Canal, at Alamo Mocho Meter Station, Mexico; $S = 0.000255$							
January 6, 1926	621.4	121.5	5.11	3.25	2 025.6	89	0.0224
January 10, 1926	680.0	121.8	5.56	3.38	2 300.0	90	0.0225
February 11, 1926	661.5	122.9	5.37	3.78	2 498.1	102	0.0195
February 1, 1926	738.8	123.1	6.00	3.53	2 605.0	90	0.0230
December 6, 1926	632.8	125.3	5.05	4.58	2 899.8	127	0.0155*
October 12, 1926	619.0	126.9	4.86	5.00	3 100.8	142	0.0135*
March 4, 1926	675.5	123.4	5.48	4.73	3 199.8	127	0.0157*
June 7, 1926	695.5	124.1	5.60	4.74	3 296.5	125	0.0160*
May 23, 1926	844.5	123.9	6.80	4.14	3 499.9	100	0.0206
August 19, 1926	729.8	124.1	5.88	4.93	3 600.4	127	0.0158*
July 15, 1926	877.5	124.8	7.01	4.21	3 699.0	101	0.0210
July 8, 1926	872.0	123.4	7.06	4.55	3 965.4	107	0.0203
(*b*) East Highline Canal, at "B" Heading; $S = 0.000322$							
January 7, 1928	187.0	54.7	3.42	2.16	405.6	65	0.028
December 25, 1928	182.5	54.5	3.35	3.25	593.5	99	0.0186
November 19, 1928	203.6	55.4	3.68	3.47	705.1	101	0.0183*
January 28, 1928	221.1	56.0	3.94	3.62	801.2	102	0.0183*
October 10, 1928	234.7	56.5	4.17	3.83	900.4	104	0.0183*
October 27, 1928	256.2	57.5	4.45	3.90	1 000.9	103	0.0185*
October 21, 1928	269.8	58.3	4.61	4.10	1 100.6	106	0.0183*
January 28, 1928	270.6	58.0	4.66	4.45	1 201.6	113	0.0175
September 4, 1928	300.7	59.6	5.03	4.35	1 301.6	108	0.0183*
May 23, 1928	298.4	59.7	5.00	4.69	1 399.7	117	0.0172
July 28, 1928	348.7	62.1	5.60	4.58	1 598.5	108	0.0183
(*c*) Central Main Canal, Boundary Meter Station, California; $S = 0.00044$; Maximum Capacity,1 050 Second-Feet							
April 13, 1928	52.40	38.08	1.38	1.94	101.85	79	0.0205
January 8, 1928	82.80	40.87	2.03	2.47	204.11	83	0.0205
March 25, 1928	120.80	43.69	2.77	3.31	399.24	95	0.0188
January 26, 1928	133.40	43.96	3.03	3.02	402.35	83	0.0218
July 1, 1928	145.70	44.83	3.25	3.45	502.87	91	0.0200*
September 21, 1928	157.70	45.49	3.47	3.81	600.14	97	0.0195*
May 4, 1928	181.80	46.80	3.88	3.87	703.83	94	0.0200*
August 4, 1928	189.40	46.79	4.05	3.96	750.69	95	0.0200*
June 8, 1928	218.2	48.71	4.48	4.15	905.84	93	0.0200
(*d*) West Side Main Canal, Boundary Meter Station, California; $S = 0.000525$							
1928	59.2	27.2	2.18	2.02	119.61	59	0.033
1928	144.2	36.7	3.93	2.76	399.29	61	0.031
1928	190.8	40.5	4.71	3.13	600.1	63	0.032
1928	225.6	41.8	5.38	3.34	751.3	62	0.032
(*e*) West Side Main Canal, Drain Meter Station, California; $S = 0.00038$							
March 18, 1929	134.00	32.65	4.10	2.26	303.22	57	0.035
January 20, 1929	136.00	29.60	4.59	2.54	345.57	61	0.033
January 7, 1929	149.60	33.90	4.41	2.68	401.62	63	0.0315
February 15, 1929	149.60	34.05	4.39	2.68	400.35	66	0.030
August 28, 1929	156.50	34.10	4.59	2.89	451.00	71	0.0275*
August 5, 1929	164.60	35.30	4.66	2.90	476.91	69	0.027*
July 12, 1929	163.80	35.95	4.55	3.06	500.48	74	0.026*
August 22, 1929	166.90	34.95	4.78	3.07	512.66	72	0.0275*
May 6, 1929	170.20	35.35	4.81	3.11	530.00	73	0.027*
May 10, 1929	172.50	36.50	4.74	3.20	552.30	75	0.027*
May 11, 1929	182.60	36.65	4.98	3.33	601.56	76	0.027*
May 21, 1929	185.30	36.85	5.02	3.34	619.92	76	0.027*
June 21, 1929	190.40	37.15	5.12	3.42	651.03	77	0.026*
(*f*) Briar Canal, at International Boundary Line; $S = 0.000382$; Maximum Capacity, 250 Second-Feet							
January 1, 1928	69.7	26.45	2.63	1.79	125.26	56	0.0300
October 30, 1928	73.6	27.75	2.66	2.10	155.0	73	0.0245
May 17, 1928	62.6	27.60	2.26	2.72	170.4	92	0.0186
April 22, 1928	59.6	27.60	2.16	3.02	180.1	105	0.0162*
June 28, 1928	72.0	27.75	2.60	2.66	191.3	84	0.0210
March 25, 1928	67.0	28.20	2.37	2.98	200.3	99	0.0177*
September 3, 1928	64.4	27.95	2.30	3.11	200.4	105	0.0165*
June 18, 1928	65.6	28.72	2.28	3.04	200.4	103	0.0168*
July 10, 1928	71.7	28.75	2.49	3.00	215.0	87	0.0198*
July 11, 1928	80.6	29.35	2.75	2.73	220.8	85	0.0207
August 7, 1928	79.0	29.50	2.68	2.90	229.2	90	0.0195*

*Sections considered non-silting and non-scouring.

listed in Table 1 (*f*) were taken. This canal should not be considered typical of those that are non-silting and non-scouring.

The action of bed silt is interesting in the fact that at times the meter gauges will indicate a smooth even cross-section and then, again, large holes are found across the bottom of the canal caused by a movement of the bed silt. It has also been noted that even where a canal has been considered self-cleansing there are times when large quantities of bed silt will move in and raise the bottom considerably. This will last sometimes for only a day or two and again for a period of several weeks. This movement of bed silt creates large fluctuations in canals of the size of the Alamo and makes it difficult to maintain a regulated flow.

From Table 1 it will be noted that these silt-laden canals of the Imperial System function under high velocities relative to earth canals. This is true in cases for discharges as low as 50% of the canal capacities. Close observations of the functioning of these canals under varying heads indicates that under flows exceeding about two-thirds of the full capacity, little trouble is experienced in the silting in or scouring out of the beds of the canals, particularly those operating under excessive grades. Relatively high velocities are required, also, to maintain movement of the bed load, which is often very erratic.

Information that is helpful in the consideration of velocities in a channel of approach to a desilting structure, whether it be to function automatically, by settling basins, or by mechanical means, might be gained from a study of Table 1. The writer is of the opinion that it would require a mean velocity closely approaching 4 ft per sec for an unlined channel of 15 000 to 20 000 sec-ft capacity, in order to keep the bed load of silt moving with a sufficient degree of uniformity. Mean velocities in excess of 4½ ft per sec would tend to turbulency. These deductions are also based on the results made from similar studies of the velocities in the Colorado River made by the Bureau of Reclamation from gauging records of the Yuma Project at the time the canal studies were made. If correct, these deductions would be adverse to the acceptance of eliminating entirely the basins in favor of under-sluice operation where so wide a channel is involved.

At the present writing (1933) engineers of the Yuma Project are conducting certain silt studies at the project diversion at Laguna Dam. Most certainly, as the author suggests, the correct solution of the problem in the design of the Imperial Diversion warrants sufficient research work along this line.

R. F. WALTER,[40] M. AM. SOC. C. E. (by letter).—The author has submitted a valuable and opportune paper, which is of particular interest to the writer due to his connection with the U. S. Bureau of Reclamation, to which organization has been entrusted the duties of designing and constructing the works discussed. The structure referred to, and for which tentative plans are shown in Figs. 5, 6, 7, and 8, has been officially designated, Imperial Dam. With its appurtenant works, it will constitute the diversion and desilting structure for the All-American Canal. The design and construction of these works now (1933) await certain legal actions and the appropriation of the necessary funds by Congress.

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[40] Chf. Engr., Bureau of Reclamation, Dept. of the Interior, Denver, Colo.

In his acknowledgment, Mr. Rothery makes reference to certain material for the preparation of Figs. 5, 6, 7, and 8 of his paper, as having been made available through the writer. The material referred to is a drawing showing a desilting plan very similar to that shown by Fig. 8 of the author's paper, but with the sand sluice omitted. Reference is also made in the paper to a cost of $3 375 000 for the proposed desilting works. Both the cost estimate and the drawing referred to by the author, are part of a report by one of the engineers of the Bureau of Reclamation on the All-American Canal investigations carried on during 1929 and 1930. The report is dated May, 1931.

While it was doubtless not the author's intention to give the impression that the plan furnished him or the estimate of cost were other than tentative, still the impression might readily be gained that they represented works designed and approved for construction. This is not the case since the plan which was the basis for the estimate is of a very preliminary and tentative nature.

The investigations leading to the report of May, 1931, were conducted in the same manner as other similar investigations of the Bureau of Reclamation. Only sufficient field surveys and office designs were made to permit of reasonably dependable estimates of cost being prepared. For such a preliminary report and estimate it was obviously not practicable to go carefully into the design of a structure of such uncertain operating characteristics. Instead, and as a basis for a tentative estimate of cost, a sketch plan was made for a structure somewhat similar in its operation features to the Laguna Dam at the head of the Yuma Canal. The desilting plan at this dam, while being far from ideal or of great efficiency, has served the Yuma Project for many years.

Reference is made by Mr. Rothery, to the methods used by the Imperial Irrigation District to prevent, in so far as possible, the bed load of the Colorado River being carried through the canal. This has proved to be an expensive undertaking and it is known that the District's officials consider even the Laguna Dam method of silt removal a great improvement over the method used by the District, and by comparison quite satisfactory and economical in operation.

Probably one of the most objectionable features of the desilting basin at Laguna Dam is the necessity of having to interrupt irrigation deliveries during the sluicing period. This is necessary because there is only one basin and no by-pass. To obviate this necessity, and in order to carry the greatly increased capacity of the All-American Canal, six desilting basins were tentatively proposed for construction at Imperial Dam. With this system the full capacity of the canal could be passed slowly through any five basins while the sixth was being sluiced of its accumulation of sludge without interrupting full canal diversion. For less than full canal capacity (which will doubtless be the usual operating condition for many years after the canal is put into operation, and even then for the greater period of each year) less basins could be utilized, but probably a better plan still would be to utilize five basins, in which case the flow through the basins would be at a reduced rate, permitting better settlement conditions. For velocities of less than 0.5 ft per sec appreciable quantities of materials that are on the border line between bed and suspended load should settle out in the basins.

Whether quantities of suspended material, such as would justify the large basins tentatively considered, will settle out with the depths necessary in the basins and at velocities of flow required, should be known definitely when the very carefully planned investigations now being carried on at Laguna Dam are completed. These investigations should demonstrate quite definitely the efficiency and economic practicability of this type of basin. Due to the availability of suitable materials—earth for the embankments and rock for rip-rap—the excess cost of this type of desilting works will not be as great as might at first be assumed, because of the long embankments required in the construction of the basins, compared with the cost of the more simple sluicing structure, shown in Figs. 5 and 6. However, should the present investigations show that no great efficiency is gained by building the large basins they should not be used.

It is not the writer's intention to enter into a discussion of the comparative merits of the desilting plan upon which the estimate of the Bureau of Reclamation was based, or of the simplified desilting structure suggested by the author, since certain important investigations being conducted by the Bureau which are expected to furnish the fundamental data for the preparation of final designs, have not yet been carried to such a point as would justify such a discussion. However, several different plans for the exclusion of silt from the canal, both bed and suspended load, have been, and are, being considered by the Bureau of Reclamation.

The general plans considered cover a wide range of construction, from the most simple sand-trap structure of unknown efficiency, to a comprehensive system of mechanical clarifiers of relatively known efficiency, such as are used in connection with municipal water purification plants and certain mining operations. The so-called vortex tube, with its various modifications, has also been given consideration for the removal of bed silt that it may not be possible to exclude from the head of the canal or that, for one reason or another, may reach the canal below its point of diversion.

The difference in first cost of the various plans considered is naturally very great and not directly comparable since the performance expected from each plan is quite different. One familiar with the silt problem of the Colorado River will realize at once that the cost of desilting works will increase rapidly with any requirement for increased efficiency in the removal of the suspended silt transported. The final choice of plan may not be based purely on first cost, or on operating cost, since the question of efficiency in operation and desirability as regards the quantity of suspended silt to be removed, may have a deciding influence in the choice.

Each succeeding year brings out more clearly the detrimental effects, to the present irrigated areas of the Yuma Project and the Imperial Irrigation

District, of irrigation with water containing large quantities of suspended and colloidal materials. Water users have been known to have closed their headgates and to do without water for limited periods, rather than to accept water especially heavily laden with fine silt. Whether it is economically possible to remove such materials or whether the construction of Hoover Dam, and possibly Parker Dam, will automatically correct this undesirable condition within a reasonable period of years, are matters for consideration.

The author's suggestion that the silt carried in suspension, during the early years of canal operation, will be of benefit to new land and lateral developments, and should be taken advantage of in connection with the development of new lands with sandy soil, may have some merit; unfortunately, however, it is not likely that any great area of the new sandy lands will have been brought under cultivation until after the stabilizing effects in the river shall have become felt and the quantity of silt carried in suspension will have been greatly reduced.

The writer is not in full agreement with the author on all the points discussed in his paper, but on one point at least he is in full agreement; that is, wherein he suggests that due to the permanence and importance of the control structure, competent investigations and research pertaining to the silt problem should be made so that as much as possible of the present guesswork will be eliminated.

In line with this suggestion the Bureau of Reclamation has already made an exhaustive research of published literature in this as well as in many foreign countries, in an effort to have available all data bearing on the silt problem. Access has also been had to many records and reports not available in published form. A bibliography of all such information has been prepared and is being added to as rapidly as new data become available.

Records have been kept of the quantities of silt carried in suspension in the Colorado River at Yuma, Ariz., for a period of more than eighteen years. Records of silt content at other points on the river are also available. These data as regards the content of suspended silt are considered quite adequate and dependable for making a comprehensive silt study. However, as regards bed load little definite information is available. Unfortunately, to date, no positive means are known to have been developed for measuring the bed load carried by streams. Study is being given this matter and plans are now (1933) under way which it is hoped will give reasonably accurate measurements of the quantity of total silt load in the Colorado River in the vicinity of the All-American Canal diversion. While positive determination of the bed load may not be possible, it is confidently believed that results can be obtained that can be used in the design of desilting works with greater assurance of reliability.

As stated previously, carefully planned investigations are being made at Laguna Dam in an attempt to learn additional facts regarding silt in the Colorado River. Analysis of the silt carried in suspension, as well as that deposited in the desilting basin, is being made. By use of the hydrometer method, determination is being made, not only of the particles susceptible of analysis by the ordinary sieve method, but of sizes down to those that require as much as two hours for settlement in still water in short tubes. It is hoped that determination can be made of the rates of settlement of the various sizes of silt particles, both in still water and at rates of flow such as may obtain in the All-American Canal, or in whatever type of desilting basin is under consideration.

A somewhat different and more comprehensive program of investigation and laboratory experimentation has already been outlined, but this can be undertaken only in case a special appropriation for the work is made available. During these investigations it is hoped to be able to ascertain with some degree of accuracy the quantity of silt that will be carried to the Imperial Dam, under flow conditions such as will obtain in the river after the construction of the Hoover and Parker Dams, and with the regulated flows required for diversion and sluicing the river channel and keeping it cleared of the sludge that will be turned back to the river from the desilting structure. Under regulated flow conditions in the river, discharges greater than 25 000 sec-ft will probably be of rare occurrence, within a few years after the completion of Hoover Dam. Discharges of less than this quantity, even with clear water released from the reservoirs, may not carry the quantity of bed load anticipated.

The author has called attention to an important problem in the development of the All-American Canal System, and one that must be given a great amount of consideration. Another problem, or rather a part of the same problem, that has been given little consideration in his paper, is that of the disposal of the sludge returned to the river from the partly desilted water diverted to the canal. This may prove to be one of the most discouraging problems of all. However, in some respects, the conditions will probably be favored by the natural occurrences in the sequence of operation of the Hoover Dam and the All-American Canal. For example, during the early stages of operation of the canal, only partial capacity will be required for the purpose of irrigation and power development along the canal. This will call for desiltation of a reduced quantity of water with a resulting decrease in the quantity of sludge returned to the river. Furthermore, a relatively large river discharge may be available, during the first few years of canal operation, for use in sluicing the sludge discharge down the river. Another factor that will also tend to diminish, possibly quite materially, the sludge return to the river below the Imperial Dam, will be the construction in the river of this dam, the crest elevation of which will probably be nearly 25 ft above the present elevation of the river bed. The dam will create a temporary reservoir of appreciable area that will act as an effective settling basin during the early period of canal operation. On the other hand, it may be that during the first few years of river re-adjustment, after completion of the Hoover and Parker Dams, relatively high river discharges will still occur and even greater quantities of bed load materials than at present will be carried to the Imperial Dam.

HARRY F. BLANEY,[41] ASSOC. M. AM. SOC. C. E. (by letter).—The author has contributed a valuable paper on the subject of silt transportation. It is of special interest at this time because large sums of money will be spent during the next few years building structures to utilize the waters of the Colorado River, and the success of such projects will depend primarily upon whether diversion works are properly designed for silt control. The paper brings out clearly the fact, not always recognized in the past, that the bed load in the river is large and that the control of bed silt is one of the major problems confronting the designer of head-works on the Lower Colorado.

In 1926, the late Samuel Fortier, M. Am. Soc. C. E., and the writer prepared a report[42] on a series of silt investigations on the Colorado River, conducted by the U. S. Department of Agriculture from 1907 to 1925. One of the surprising facts revealed in these studies was that the suspended silt load in the river at Topock, Ariz., was considerably greater than that at Yuma, Ariz., 206 miles farther down stream. Comparison of the normal annual load at Topock with that at Yuma, exclusive of the Gila River, indicated that 36% of the silt at Topock either passed Yuma as bed silt or was temporarily deposited in the channel or flood plains above Yuma.[42] An investigation by Mr. C. S. Howard[43] confirms this finding. The average of his results for a 3-year period ending September 30, 1928, shows that the annual suspended matter carried by the river was 286 000 000 tons at Grand Canyon, 231 000 000 tons at Topock, and 174 000 000 tons at Yuma. After applying a correction factor by estimating the inflow of the Gila River above the Yuma Station, these data indicate that about 34% of the suspended silt load at Topock became bed silt before reaching Yuma. The difference between the average annual quantity of suspended silt measured at the Grand Canyon and that at Yuma, exclusive of the Gila, is estimated as 72 000 acre-ft, based on the assumption that 12% of the silt load at Yuma comes from the Gila River,[42] and that the dry weight of silt is 85 lb per cu ft. Undoubtedly, most of this becomes bed silt.

Thus far, no method has been developed by which the volume of material moved as bed load can be measured. The aforementioned calculations may be a means of approximating the quantity of bed load carried by the river past the diversion works contemplated for the All-American Canal. The writer is of the opinion that as the river emerges from the canyon section, it is carrying its greatest load of suspended silt due to the steeper grades, higher velocity, and churning effects of the canyon section; then as the river flows on flatter grades below the canyon section the heavier silt is temporarily deposited and becomes bed silt, or rests on the bed of the channel until a flood carries it farther down stream. Many estimates have been made of the volume of silt transported by the Colorado River and some of them are shown in Table 2.

The author gives the reader the impression that "the traveling suspended load cannot be greatly reduced with the desilting basins." It is true that

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[41] Irrig. Engr., U. S. Dept. of Agriculture, Los Angeles, Calif.

[42] "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier and Harry F. Blaney, *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, 1928, p. 53.

[43] "Suspended Matter in the Colorado River in 1925-1928," by C. S. Howard, *Water Supply Paper 636-B*, U. S. Geological Survey, p. 24.

TABLE 2.—SOME ESTIMATES OF SILT TRANSPORTED ANNUALLY BY THE COLORADO RIVER

References	Location	Period	Dry weight of silt per cubic foot, in pounds	ANNUAL SILT LOAD, IN ACRE- FEET		
				Suspended	Bed	Total
Dole and LaRue[*]	Yuma, Ariz.	1895–1914	93	80 000	......	....
Mead, Schlecht, and Grunsky[†]	Yuma, Ariz.	Average	100	90 000	12 000	102 000
Weymouth[‡]	Yuma, Ariz.	1909–1922	86	105 000	......	....
Davis[§]	Yuma, Ariz.	Average	85	113 000	......	....
Davis[§]	Boulder Canyon.	Average	...	88 000	......	....
Rothery[‖]	Yuma, Ariz.	1912–1921	86	94 800	......	....
Fortier and Blaney[¶]	Yuma, Ariz.	Average	...	119 000	19 000	138 000
Fortier and Blaney[¶]	Boulder Canyon.	Average	85	....	......	137 000
Grunsky[**]	Yuma, Ariz.	1911–1927	100	94 500	......	....
Grunsky[**]	Yuma, Ariz.	Average	100	....	25 000 to 30 000	....
Howard[††]	Grand Canyon.	1925–1928	86	152 700	......	....

[*] *Water Supply Paper 395*, U. S. Geological Survey, 1916, p. 222. [†] Report of All-American Canal Board, 1920, p. 25. [‡] "Colorado River Development", Senate Doc. 186, 1929, p. 52. [§] "Problems of Imperial Valley and Vicinity", Senate Doc. 142, 1922, pp. 3-4. [‖] *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1419. [¶] "Silt in the Colorado River and Its Relation to Irrigation", *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, 1928, pp. 61–62. [**] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), pp. 1118, 1126. [††] "Suspended Matter in the Colorado River in 1925–1928", *Water Supply Paper 636–B*, 1929, p. 28.

desilting has not been so successful at the Rockwood Heading of the Imperial Irrigation District, due primarily to the fact that the river is not regulated and practically the entire flow is diverted into the heading at extremely low stages of the river. However, experiments made in 1918 by the Bureau of Agricultural Engineering on the efficiency of desilting at the head-works of Laguna Dam, indicated that the desiltation ranged from 33 to 72% and averaged about 57 per cent.[44] Raymond A. Hill, M. Am. Soc. C. E., also carried on experiments at intervals during several years, and he found that desiltation ranged from 18 to 70% with an average of 50 per cent.[44]

While the intake at Laguna Dam for the Federal Project at Yuma, is the most effective that has been installed to date on the Colorado River, in ridding the water of its heavier silt it is creating a soil problem that may be difficult to solve. By the desilting process the clays and colloidal material are separated from the sand in the silt and deposited on the irrigated land. This annual deposition is bringing about a change in the character of the surface soil, making it more sticky, more difficult to cultivate, and more impervious.

The writer believes that the term, "silt," is preferable to the term, "soil," in the title of the paper, since the term, "silt," is now more commonly used by engineers and is subject to less criticism by soil authorities than in the past.

CALVIN V. DAVIS,[45] M. AM. SOC. C. E., AND DIRK A. DEDEL,[46] ASSOC. M. AM. SOC. C. E. (by letter).—This paper describes excellent designs for structures to remove silt and bed load, which are appurtenant to the proposed Colorado

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[44] "Silt in the Colorado River and Its Relation to Irrigation," *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 58.

[45] Chf. Designer, Ambursen Dam Co., Inc., New York, N. Y.

[46] Designing Engr., Ambursen Constr. Co., New York, N. Y.

River diversion dams. In view of the thoroughness with which the author has treated all phases of this problem, it is difficult to suggest basic improvements in his tentative plans. Several individual elements of the plans, however, may be discussed to advantage, owing to the fact that alternate designs capable of performing the same functions may be used.

In Fig. 6 (*a*) and Fig. 8 (*a*), the author has shown, up stream of the diversion works, intake tubes that consist of a floor and a horizontal partition supported by light vertical walls. These intake tubes are intended to convey a major portion of the traveling bed load to conduits which either pass directly under the dam and flume and discharge in the river down stream of the flume, as shown by Fig. 6 (*a*), or turn at right angles at the toe of the intake structure and discharge in the river below the main diversion dam, as shown by Fig. 8 (*a*).

The writers are of the opinion that while the intake tubes, as shown in Fig. 6 (*a*) and Fig. 8 (*a*), will trap most of the moving bed load there is the possibility that under certain flood conditions a part of this bed load may pass into the canal.

A modification of Mr. Rothery's intake design which the writers believe will, under all flood conditions, eliminate the traveling bed load and most of the suspended silt load before it reaches the canal, is shown by Fig. 9, which is a draft distributor devised by Mr. R. D. Johnson and the late Petrus Wahlman, M. Am. Soc. C. E. This modification merely consists of substituting a slot of the type shown in Fig. 9, in the top of the intake tube, for the horizontal partition and rectangular entrance as indicated by Mr. Rothery. The tube shown by Fig. 9 (*b*), is connected to a bed-sand conduit at the heel of the intake dam. This conduit is carried across to the heel of the dam in the main river section where its discharge into the river is regulated by sluice-gates (Fig. 9 (*c*)).

This arrangement would draw in from the top the entire bed load of sand, and most of the suspended load, with a minimum of disturbance. A model test,[47] reported by H. G. Acres, M. Am. Soc. C. E., indicated that water may be drawn into this type of tube in shallow rivers with practically no disturbance. The velocity through the slot, however, would be more than sufficient to draw in the traveling load of sand and most of the suspended silt. An additional advantage of this type of intake tube is that it eliminates all danger of river-bed erosion such as might occur under certain conditions at the mouth of a tube wth a rectangular opening.

If desilting basins are used the writers believe it more advantageous to locate the bed-sand conduit at the heel of the intake dam (such as shown by Fig. 9 (*b*)), rather than to locate it at the toe of the intake dam as shown by Fig. 8 (*a*) of the author's paper. In the arrangement shown by Fig. 9 (*b*), the intake dam and conduit are independent structures. There can be no uplift pressure under the crest and apron slab from the head-water and the flow in the bed-sand conduit does not come in contact with the buttresses and piers of the intake structure as it does in Mr. Rothery's designs. It seems to the writers that there is considerable advantage in keeping separate structurally, as far as possible, those parts of the intake that perform different functions.

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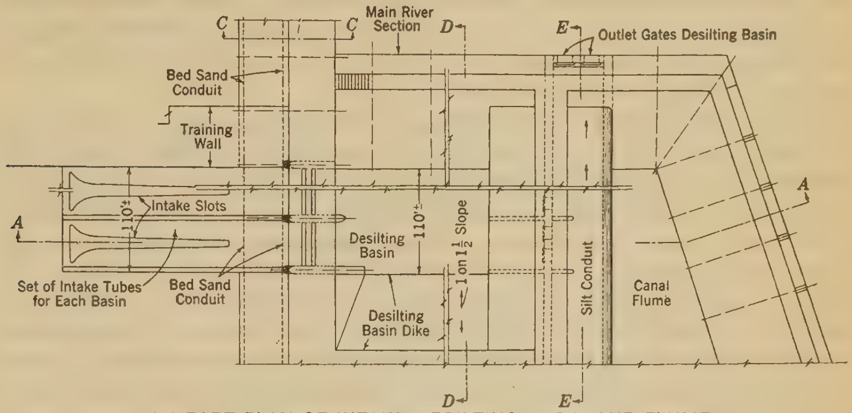
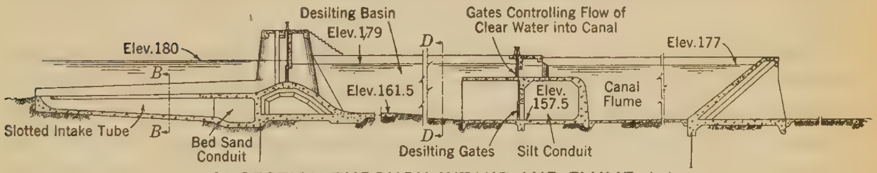
[47] *Engineering News-Record*, September 10, 1931, p. 403.

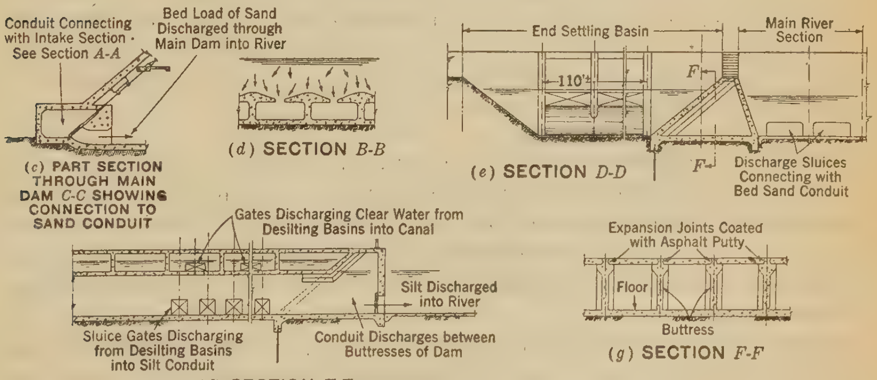
The writers agree with Mr. Rothery's finding that the traveling suspended load cannot be greatly reduced with desilting basins. It is believed that the use of draft-distributor intakes (as described in the foregoing) would make unnecessary the use of these expensive basins. Further research along this line would be highly worth while and might result in large savings in cost.


C
C
Main River Section
D
E
Outlet Gates Desilting Basin
Bed Sand Conduit
Training Wall
110'±
Intake Slots
A
A
Desilting Basin
110'±
1 on 1½ Slope
Silt Conduit
Canal Flume
Set of Intake Tubes for Each Basin
Bed Sand Conduit
Desilting Basin Dike
D
E

(a) PART PLAN OF INTAKE DESILTING BASIN AND FLUME


Desilting Basin
Elev. 179
Gates Controlling Flow of Clear Water into Canal
Elev. 180
D
Elev. 177
B
Elev. 161.5
Elev. 157.5
Canal Flume
Slotted Intake Tube
B
Bed Sand Conduit
D
Desilting Gates
Silt Conduit

(b) SECTION THROUGH INTAKE AND FLUME A-A


Conduit Connecting with Intake Section · See Section A-A
Bed Load of Sand Discharged through Main Dam into River
End Settling Basin
Main River Section
110'±
F
F
Discharge Sluices Connecting with Bed Sand Conduit
Gates Discharging Clear Water from Desilting Basins into Canal
Silt Discharged into River
Sluice Gates Discharging from Desilting Basins into Silt Conduit
Conduit Discharges between Buttresses of Dam
Expansion Joints Coated with Asphalt Putty
Floor
Buttress

(c) PART SECTION THROUGH MAIN DAM C-C SHOWING CONNECTION TO SAND CONDUIT

(d) SECTION B-B

(e) SECTION D-D

(f) SECTION E-E

(g) SECTION F-F

FIG. 9.—PROPOSED MODIFICATION, DESILTING BASIN AND FLUME.

If desilting basins are used, however, there are several possible modifications of the author's designs which might result in both improved operation and savings in cost.

As Mr. Rothery has pointed out the slightest movement in a prism of water that has been stilled for the purpose of settling out the silt, will put this silt back in suspension. The gates discharging the water from the desilting basin to the flume (Fig. 8 (*a*)) might cause sufficient disturbance when in operation to put some of the sludge in suspension again even if these gates are in the most favorable position.

The writers believe that the alternate design for an outlet structure, such as is shown in Fig. 9, will eliminate any possibility of disturbance in the settling basin while the outlet gates are operating. This structure consists of a silt conduit from which the sludge from the desilting basins is discharged laterally into the river. The flow of this sludge into the silt conduit is controlled by gates leading from each basin at the bottom of this conduit. The flow of clear water from the desilting basin to the canal is controlled by a small gate structure on top of the conduit. In front of these gates are horizontal partition slabs supported by light vertical walls which are somewhat similar to those used by Mr. Rothery in front of the intake tubes. These horizontal slabs are intended to prevent the higher velocity adjacent to the gate from causing any disturbance in the desilting basin after the sludge has settled.

The flow through the silt conduit is controlled by outlet gates in the outer partition wall of a dam of the buttress type supported by a floor-slab. Two of the buttresses of this dam are continuous with the walls of the silt conduit, as shown by Fig. 9 (*a*) and Fig. 9 (*f*). This type of buttress dam has been shown for both the end of the desilting basin and the sides of the canal flume.

It is believed that the arrangement of the silt conduit shown by Fig. 9 will be more enonomical than the proposal to discharge the sludge from the desilting basin through conduits under the flume, as shown by the author. The expensive conduits are omitted, and only light paving is required for the bottom of the flume.

THADDEUS MERRIMAN,[48] M. AM. SOC. C. E. (by letter).—The subject of this paper is of great importance, and the author has forcefully presented the situation with respect to the "bed load." In 1930, the writer suggested[49] the view referred to by the author that the suspended load is only a part of the total and that the bed load carried by a flood is proportional to the bed volume scoured out.

Further reflection during the succeeding three years and a careful reading of the author's paper, including the discussions thereon, confirms the previous conclusion of the writer that,

> The bottom load carried per unit of time is equal to the volume of material scoured out per unit of length.

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[48] Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

[49] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1142.

This theorem is a statement of the conditions that must obtain if the river channel is to refill itself while the flood is subsiding. Otherwise, the regimen of the river would be subject to constant and progressive change in proportion to the amount by which the actual conditions differed from those of the equilibrium which the theorem defines. The stability of the regimen between Black Canyon and Yuma is, therefore, evidence of the correctness of the proposition which, in connection with soundings and under the limitations stated[49] by the writer, furnishes a means of approximating the "bed load."

The completion of the Boulder Canyon Reservoir will greatly upset the equation of equilibrium in that no new material will be available to pass on down the river. Under the new conditions of reduced flood stages and uniformity of flow the regimen of the Colorado will at once begin to show marked changes which are not readily predictable, but which will be of the nature indicated by the author.

J. C. Allison,[50] M. Am. Soc. C. E. (by letter).—This paper is extremely interesting. Many articles and studies have been produced dealing with this subject. The excessive heights to which the great canals of the Imperial Valley are now elevated above the surface of the farming land, due to the deposit of silt, bear witness to the extreme importance of the problem. The financial statements of the Irrigation Districts of the Valley reveal the millions of dollars in operating cost traceable in greater part to the silt-removal items, and these tabulated millions are perhaps only a fraction of the grand total of costs to the farming community as a whole, chargeable to this one item. The writer has witnessed the progressive building up of the spoil banks along the canals with a continued sense of bewilderment as to what might be the final solution.

Exhaustive studies by the United States Bureau of Reclamation will undoubtedly reveal the best practical solution of the problem. Until these studies are completed it is difficult for an engineer to determine a plan that will come nearest to solving all the vexing questions that the "New Deal" on the Colorado River, with the Boulder Dam in partial control, has brought to the front. Already Mr. Rothery's paper has brought out what are perhaps the major questions to be solved. The writer's experience with this particular matter, prompts him to state the three major questions to be answered, all with the hope that the suggestion may be helpful in guiding the Bureau of Reclamation and the profession to the best practical plan.

I.—The greatest problem of all is the disposal of the sludge returned to the river from the partly desilted water diverted to the new canal at the proposed new Imperial Dam. In the writer's opinion, the volume of soil yet to be moved by the Colorado River below Boulder Dam can be measured only by the aproximate area of river soil deposited in the valleys of the Colorado between the mesa walls. This is limited, of course, to the uncontrolled sections of the stream bed. As the partly regulated stream seeks to establish its ultimate gradient and final position under the new operating conditions artificially set up in the creation and operation of the dam and its power plants, an unknown volume of suspended silt and bed load will continue to move down stream.

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[50] Cons. Engr., San Diego, Calif.

If one can picture a gigantic task of finally disposing of all this deposit through the several hundred miles of unregulated channel between Boulder Dam and Imperial Dam in generations to come, the uppermost thought in one's mind is, what shall be the final resting ground for this uncalculable yardage? The answer is that there are only three ultimate places of rest large enough to accommodate the ultimate load: The Gulf of California, the Laguna Salada Basin, at the extreme lower end of the river, or the Salton Sea, in Imperial County, California. To move the sludge to any of these basins entails an expenditure of great volumes of transporting water. If the ultimate irrigation demand does not preclude this great expenditure of sluicing water, then Mr. Rothery's plan of selective separation of the lower third of the waters transporting the bed load, becomes one worth while studying and perfecting if possible.

On the other hand, if an economy in water is necessary in providing a sluicing head, then such a plan as is proposed by Mr. Rothery is barred from consideration. As a matter of fact, the entire plan of silt removal at the head of the canal, or at any point along the canal where the sludge is to be returned to the river bed, must be barred from consideration unless sufficient water can be supplied over and above the irrigation demand, to assure a volume and velocity sufficiently great to carry the bed load on through to one of the basins of rest mentioned.

There is apparently lurking in the author's mind the possibility that through some plan or other, these sluicing waters can be picked up and used farther down the river for irrigation below the Imperial Dam. The Mexican lands of the Lower Colorado Delta present the only area upon which this water could be used. Such a water supply, loaded to its maximum carrying capacity with bed load sand and silt, would not only be an unacceptable gift to Mexico but, if interrupted in its flow by diversion works in Mexico, would defeat the very purpose of its release at Imperial Dam.

The real answer to this particular problem lies in the determination, by the Bureau of Reclamation, of the quantity of water required annually to clear the river channel of its accumulation between Imperial Dam and the basin of final rest for the material. The surplus of water available during the early years of operation, before the period of full irrigation demand, cannot be considered and counted on too seriously in the design of the desilting works. The permanent volume of water available for sluicing purposes over and above the ultimate irrigation demands must be the quantity counted on to rid the river channel of its accumulated volume of sludge, because the bed load of sand picked up below the Boulder Dam and contributed to the channel by the drainage areas of Arizona and California tributaries to the river, will supply a load to be transported to the canal works of the Imperial Dam long after the peak irrigation dam is reached.

The plans for Boulder Dam provide a continued firm horse-power output of about 663 000 which will produce the 4 330 000 000 kw-hr of prime energy contracted by the Government for sale. Based on 83% efficiency and 10% maximum shortage, as estimated by the Bureau of Reclamation, this output will require an average volume ranging between 14 000 cu ft per sec and 17 500 cu ft per sec diverted continuously through the Boulder Dam power plant. According to the writer's calculations, the peak irrigation demand for the 2 100 000 acres finally to be reclaimed below the dam, will utilize this entire output from the power plants in the summer. In the winter, irrigation of the desert valleys of the Colorado River below the Boulder Dam will only use one-third of this power-development output of water; yet, unless and until other great dams in the canyons of the Colorado River are constructed, the irrigation demand cannot be equated with the power demand. Firm power in the aforementioned amounts as contracted for must be continuously developed in order to pay the costs of the dam within the 50-year period provided by law. Therefore, two-thirds of the winter output through the power plants must be continuously wasted. According to the writer's calculations, this seasonal wastage will amount to 3 544 000 acre-ft annually at Imperial Dam. The investigation being undertaken by the Bureau of Reclamation should determine whether or not this seasonal wastage is sufficient to clear the annual deposit of sludge from the river channel below Imperial Dam permanently. If it will not, the plan of utilizing the river bed as a depository for the bed-load soils taken from the All-American Canal, should be abandoned, except in the event of a re-arrangement of irrigation water delivery to Imperial Valley, as described under Heading III herein.

II.—Except for the "impatience and sentiment from the large agricultural communities of Imperial Valley," as pointed out by Mr. Rothery, it is doubtful whether the removal of all the silt in suspension will be found by the Bureau of Reclamation to be entirely justifiable, even if a plan can be devised to do so economically. When applied to the porous lands of the eastern mesa the clear water will not be as practicable, in canal operation or irrigation, as the silted water; when delivered to the lateral canals on the steep contours of the eastern and northern sections of the Imperial Irrigation System, the clear water will bring about compensative adjustments of canal grades, requiring many additional structures or concrete lining of the canals; also, more adequate drainage systems than are provided for in the existing financial plans. In the central and west central section of the Imperial Valley, where the canal grades are generally already too flat, a clear-water delivery would be extremely justifiable and beneficial if accompanied by adequate drainage provisions. A delivery of uniformly clear water for all sections of the irrigated lands to replace the silt-bearing waters abruptly, is not practical. It remains again for the Bureau of Reclamation to determine through its intensive study the extent to which the suspended soils can be removed justifiably. It appears to the writer that a less ambitious policy of suspended silt removal should be undertaken at the beginning, with provisions for later enlargement and perfection in various sections of the canal system, after a "cut and try" experience for a few years.

III.—The writer has in mind the great basin known as the Laguna Salada Basin in Mexico in the lower end of the Colorado River, as a final resting ground for the silts and bed sands of the river, rather than either the Gulf of California or the Salton Sea Basin. This selection is made because here all the sluicing waters used in clearing the channels above, can be recaptured and used for irrigation. As a matter of fact, with the establishment of a terminal reservoir at the Laguna Salada, no part of the river water will find its way to the ocean, because all of it can be recaptured through every month of the year except at times when more than 75 000 cu ft per sec are released at Boulder Dam.

This discussion is not the proper place to review the technical problems involved in establishing this reservoir, any more than to describe its characteristics briefly. At the extreme lower end of the river, just before the channel enlarges to form the upper area of the Gulf of California, the high waters of the river sometimes overflow and back up to the northward into the Laguna Salada, an extensive basin separated from the main delta of the Colorado by the Cucopah Mountains. The southern entrance to this huge basin is narrowed down to 5 or 6 miles. The basin broadens to the north between the Cucopah Mountains on the east and the Juarez or Peninsula Range on the west, finally terminating against a high mesa formation 8 or 10 miles south of the International Boundary Line. The floor slopes from 7 ft above sea level at the south to 17 ft below sea level at the north. This basin has been studied thoroughly, and it has been found that by blocking the southerly opening with a dike averaging 25 ft high, and by tunneling through 4 miles of the mesa formation at the north, a huge reservoir can be created, with drainage wasteways both at the south into the Gulf of California and at the north into New River and the Salton Sea. When the floor of the reservoir is desalted and the area reduced and deepened to save unnecessary loss of water by evaporation, the reservoir will still have a holding capacity of from 4 000 000 to 5 000 000 acre-ft of water.

A flood channel connecting the river through its delta with this reservoir entirely by gravity can be built by the two Governments under their agreed plan of international flood control on the lower river. This can be done with far less expenditure of money than such a channel can be built to the Gulf of California either along its present broken course, or along its abandoned channel to the east. The effect of this work would be to capture and store in Laguna Salada Reservoir all the water that will otherwise waste into the ocean. After the Boulder Dam is completed, this supply of water that cannot be regulated and saved by other means will amount to about 5 000 000 acre-ft per year. This wastage must continue until other dams the size of Boulder Dam are built in the upper canyons of the Colorado, in order to equate the power demand with the irrigation demand. Should all the water be saved by the simpler process of allowing it to flow by gravity into the Laguna Salada Reservoir, many millions of dollars will be saved, as other more expensive dams above will then not be necessary to save the winter discharge through the power plants for summer irrigation needs.

By this same process all international complications can be eliminated. The reclaimed waters can be delivered into the All-American Canal for all lands elevated in Imperial Valley below the sea-level contour. In a minor degree they can be used on Mexican lands along the border by gravity, where the elevations are not higher than 15 ft above sea level. The establishment of the reservoir would probably be the controlling factor in settling the question of the quantity of Colorado River water that Mexico is to have. As a matter of fact the recapture of waters, which otherwise would run perpetually to the ocean, will yield a supply more than sufficient to fill Mexico's demand if an exchange can be made of these waters for supplies delivered to Mexican lands at some point on the border higher than the 15-ft contour.

The cost of creating the reservoir with the delivery channel into it, the waste channels at both ends, and its delivery conduit northward, including the cost of removing the salt from the floor of the basin, could be met through a charge of 25 cents per acre-ft for the water delivered from this storage through a period of 50 years. With its creation, no economy in the use of channel-sluicing waters at any time of the year would be necessary at Imperial Dam. As the floor elevations are raised through the deposition of silt, in the reservoir, the water level would be raised correspondingly. Due to the excess grades in the river channel constructed to the reservoir, this raising could go on continuously by raising the channel banks and the supporting ground along the entrance channel with the silt dredgings taken from the channel. The very size of the reservoir itself, almost completely surrounded by mountains and high mesa plateaus, affords a suitable resting ground for Colorado silt, in all the years to come.

S. L. ROTHERY,[51] M. AM. SOC. C. E. (by letter).—The questions cited by Mr. Grunsky, as to the physical characteristics of the Colorado River silts, need to be answered by research as a first step in attacking the problem, the magnitude of which, he states, is unquestioned. Acceptance of a difficulty without seeking its analysis is contrary to engineering principles. Suggested answers to unexplained riddles often present themselves to the questioning mind, but the light of research is wanted in order to reject or verify suggestion.

The riddle of the turbid flow problem appears to be: Why do the suspended silts maintain general uniform distribution through a flowing cross-section, the charge or percentage of load remaining practically unchanged to any distant down-stream point?

A suggested answer may be reasoned from the studies before referred to[22] contrasting the shapes of silt particles to those of sand. Professor Terzaghi states that even a high-powered miscroscope does not discern the constituents of "micro-mud" (fine silts), but claims that their scale-like property is supported by many proofs, some of which are stated in the aforementioned studies. It is somewhat disappointing that none of the discussers gave new light on the physical difference between silts and sands, particularly as regards their carriage in stream flow and their characteristics of deposition in quiescent water. Partial settlement will be of little help in removing the turbid appearance in the first years following the storage behind the Boulder Dam; however, the relative volume of finest silts remaining in the 250 miles of river channel down stream, is so small in comparison to the present annual suspended load, that the question is again raised as to whether great desilting basins are really required.

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[51] South Pasadena, Calif.

In explanation of the small remaining volumes, it must be remembered that the origin of the fine silts is above the Boulder Reservoir, a function of which is to trap the present annual quantities of 90 000 to 110 000 acre-ft of suspended silts. The Imperial Diversion will be 293 miles below the reservoir, and the river-channel area in this length is about 31 000 acres, according to U. S. Geological Survey Topographic Sheets. The effect of raising the water surface at the diversion structure and also at Parker Dam, will be to subject only a part of this length and consequent area to the degrading process, before the ultimate new gradient is attained, say, to 250 miles and 25 000 acres. Should the ultimate maximum recession be 25 ft, the mean at $12\frac{1}{2}$ ft would give a product of 312 500 acre-ft to be removed in, say, 10 years, one-third (34%) of it being the "fines". Then, again, since only one-fourth of the annual flow is diverted, the future turbid problem appears to become so greatly reduced that it seems reasonable to expect a noticeable improvement in the clarity of the flow soon after the Boulder Reservoir is in operation.

A careful investigation of the silts in the flowing canals was undertaken by the Department of Engineering of the State of California in 1918–19.[52] Quoting from this report:

> "By far the greater part of the total silt in suspension in the Colorado River and the irrigation canals is fine material that passes the 280-mesh per inch screen, * * * this material was in all cases found to be equally distributed throughout the vertical for all velocities under which tests were made, including mean velocities of less than one-half foot per second in small ditches. * * * The silts of the Colorado appear to be comparable to those of the Nile in degree of fineness, but they are much finer than those of the rivers of India."

Silt research on the Nile,[53] showed that in the lowest one-third depth, where the river was 26 ft deep, about one-half the coarse (that collected on a No. 100 sieve) and of the fine (that collected on a No. 200 sieve) sands, were located. It was certainly not the writer's intention to infer that there was a definite line of demarcation between the silts and sands in the turbulent floods of the Lower Colorado River; but, in the expected future regulated flows, when the maximum discharges will be from one-half to one-third their former volumes, and where the stream regimen can be improved to avoid turbulent approach, most of the sands in transportation will be near to, and along, the bed, which will be about 12 ft below the water surface. If a structure divides the water such that the sands in the lower one-third of the stream section will be carried on through under-sluices, those grains diverted to the canal with the upper two-thirds, will amount to only a minor percentage of the entire sand load, and this percentage could be eliminated in sand-trap structures at appropriate wasteways.

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[52] Seventh Biennial Rept., Dept. of Eng., State of California, pp. 117–120.

[53] *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI, pp. 205–206.

The tributary streams and washes in the 293 miles will occasionally be a source of additional sand, and perhaps some fine silts, to the river. However, the discharges from cloudburst rains on these tributary water-sheds are far from being continuous inflows, and it is believed that the contributed water is only the equivalent of a few days' average flow of the main river.

Mr. Rouse believes that the desilting basins should be built in addition to the desanding works, and suggests certain locations other than at the diversion structure. In this consideration (and always having in mind the time of detention of the flow to precipitate the fine silts referred to in the foregoing quotation, which from years of experience on the delta is emphatically endorsed), the writer offers the following plan as best adapted to obtain a maximum clarity of water for the present ranchers and the beneficial effect of silty flow for the new lands to be reclaimed on the East Mesa:

(*a*) Obtain the best possible desanding by under-sluices in the diversion structure, and perhaps a sand-trap structure across the canal not too far distant from the main structure, making no attempt to precipitate the suspended silts;

(*b*) Construct separate stilling-basins for the Yuma Project, to desilt 1 500 sec-ft, at a suitable location to the head of the Yuma Main Canal and where the sludge can be sluiced to the river, probably somewhere in the vicinity of Station 250 on the survey; and,

(*c*) Construct separate stilling-basins for the present irrigated lands of Imperial Valley, to desilt 5 000 (perhaps 7 600) sec-ft at about the west edge of the East Mesa, say at Station 2 990, sluicing along an obtained right of way to the Alamo Channel.

Then, whatever degree of desilting is attained by the type of basins devised, the problem is much simplified, due to the lesser volumes, than if one were seeking to detain 10 000 to 15 000 sec-ft, which has a velocity of $4\frac{1}{2}$ ft per sec; and a greater efficiency of desiltation could be accomplished at a minimum of construction and of daily operation costs. Furthermore, the long main canals crossing the porous soils to the East Mesa and on the Mesa, would receive the benefits of silty flow to help minimize percolation losses and contiguous areas of seeped lands.

It is true that an extra few hundred second-feet of sluicing water must be conveyed to the locations. The objection to the sludge going to Salton Sea is of growing lesser importance, since it has been argued that the suspended load is a decreasing one yearly and there is a relatively small total volume of the "fines" to be removed below the Boulder Reservoir.

Should the present lands of Coachella demand clear water, a smaller pondage could be provided near Salton, Calif., sluicing to the Salton Sea at about Station 4 200 on the Coachella Canal where the velocity is only 2.7 ft per sec.

The writer admits, and expressly stated in the paper, that the figures used to present a viewpoint of the bed load, were necessarily assumptions. His claim to being conservative in the volumes deduced can be re-stated, since in one instance a scour depth of only 9 ft was used, and, in the second instance, only 50% of the 300 miles in length were taken to allow for "deeps" and alternate "shoals". Both instances produced a removal (during a few weeks' time) of soil greater than the estimated yearly suspended load.

When excavating for the foundation of Boulder Dam, where the low-river water surface is at Elevation 645, a piece of sawed 2 by 6-in. board was found firmly embedded in gravel at approximately Elevation 595. The difference of 50 ft is indicative of deep scour. Reference has been made to the deep scour at Yuma,[13] and the writer recorded[54] a scoured depth of 20 ft on the delta on a rising flood, when the discharge was only 60 000 sec-ft and when the metering cableway was swept away in the flood. This flood increased to 110 000 sec-ft without any further raising in elevation of the water surface, indicating a still greater depth of scour. It must also be conceded that the distance of 300 miles is shorter than that in which the scoured material has been excavated and removed by the gradual rise of the spring flood.

The writer is not alone in pointing to the indications of this voluminous bed load. In 1930, Thaddeus Merriman, M. Am. Soc. C. E., stated[55]:

> "In any event, indications are that the volume of this [bed] load, when determined, will prove to be substantially greater than the suspended load which is now known within reasonable limits."

Mr. Merriman now confirms his previous conclusion. Mr. Blaney also makes a reference to the fact that there has been lack of recognition of the greatness of the traveling bed load. Such statements are of great value and will help engineers to realize the magnitude and the problem of the bed load, which the writer has endeavored to feature a number of times since 1921.

It was not the writer's intention that his paper should have been taken as a criticism of the tentative plans of the Bureau of Reclamation. During years of experience on the delta, he observed instances of scour, of deposition, and of suddenly changed river courses that seemed incredible to account for, until familiarity with the region and its disemboguing mud torrent provided a logical reasoning for what at first were surprises. He is convinced that the valuable estimates of soil transported as "silt in suspension", comprise only a part of the story. The silt problem of the Lower Colorado River has always been of absorbing interest, which has been responsible for much personal research and deduction. In submitting this paper, spontaneously written in a few days, the writer now believes that his presentation as to (*a*) a large bed load, and as to (*b*) the feature that its removal can be effected with the usual under-sluices while it is practically impossible to remove the turbid condition of the flow for great volumes of moving water, stress the fundamental principles of the problem.

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[54] *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1417.

[55] *Loc. cit.*, Vol. 94 (1930), p. 1144.

Few engineers experienced on the Lower Colorado River have voiced their findings of silt characteristics. Realizing the importance of the subject for determination by specific research, in the "Synopsis" of the paper, the writer invited the publication of such research and the pooling of available knowledge. It is refreshing to know of the important investigations now in progress, referred to by Mr. Walters.

The psychological effect of expensive desilting basins is promissory of clear water to the canal systems; the writer had endeavored to show that this result is economically unattainable, since turbidity is produced by the "fines" (silts passing a 200-mesh sieve, and not susceptible to rapid deposition). The excessive "muddiness" of the water is the reason for the so-called desilting basins; yet only a fractional part, if any, of these excessive volumes of "fines" would be removed by the basins.

The reduced silt content, referred to by Mr. Blaney as about 50%, at the Laguna Dam Diversion to the Yuma Project, is suggestive that the multiple-basin plan will be equally effective for a diversion ten times greater. This may or may not be true. As the "fines" are not removed by the retarding basins, it may be preferable not to remove the coarser silts in carriage in suspension, in order to help guard against producing the objectionable sticky cultivating soil developing on the Yuma Project. In comparing efficiencies of proposed desanding basins with those of suggested under-sluices for eliminating sand in the approaching water cross-section, it would seem important and desirable to consider this precaution, the ideal requirement being to eliminate such sufficient part of the traveling sand as will suffice to prevent sand blocks occurring in the Main Canal. This is probably a matter of practical judgment from noting similar conditions and treatments on other great world projects.

The Sarda Canal, in Northern India, designed to carry 9 500 sec-ft, takes off from the Sarda River which "carries immense quantities of coarse silt during the flood season, and it was feared that in spite of all precautions, a considerable quantity would pass into the canal."[56]

The Sarda diversion structure is a side offtake "consisting of 16 bays of 20-foot spans. There is a masonry sill curved up stream to give a stream-line entry and thus avoid eddies which might cause silt to be picked up from below." The contiguous river sluicing sill or apron is at a depth of 6.75 ft below the masonry sill. Behind the sill are rising steel gates, which are usually lowered to permit the passage of top water over them into the canal. It is reported[56] that the canal head is working admirably; it excludes all heavy silt, and only the fine silt in suspension, which is useful for forming water-tight berms, is carried into the canal.

If eddies can thus be prevented by combining a stream-line entry design to the sills of the diversion gates, the horizontal partition may be shortened in length to its remote up-stream edge. It has been stated that the under-sluice passages would choke with submerged driftwood. Analyzing this possibility, the Boulder and the Parker Dams will trap such wood, and any occasional pieces brought into the approaching stream by intervening tributaries with cloudburst run-off, could be found and removed by the regular inspection and duties of the gate operators. Access to the passageways must be provided in the design; a tentative idea is suggested in Fig. 10.

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[56] *Minutes of Proceedings*, Inst. C. E., Vol. 23, 1931-32, pp. 7-8.

Mr. Collings has estimated that 4 700 sec-ft of sluicing water would be the flow through the suggested under-sluices, and believes that 3 000 sec-ft would be adequate if the multiple basins were adopted. The flow through the under-sluices should be controlled by outlet gates, and would be variable depending on the traveling bed load and on the seasonal demand for the diverted water.

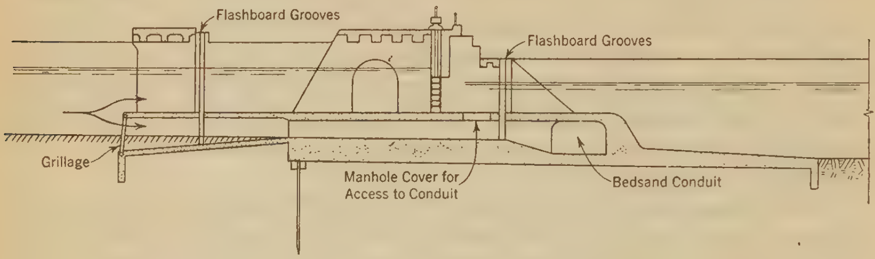

Fig. 10

For one or more decades after the Boulder Reservoir is first put in operation, a major part of the annual flow of the river will pass the diversion structure. The Yuma and California projects will use about 5 000 000 acre-ft annually, while the yearly run-off of the river varies between 9 000 000 and 25 000 000 acre-ft. Therefore, the question as to whether a few thousand second-feet of surplus water can be made available for the requirements of sluicing flow need not be raised for many years; nor will the market for the potential hydro-electric power be developed toward its ultimate capacity for a long period, to require conservation of all available water.

The writer purposely refrained from venturing into the field of probable effects that may be produced in the lowest reaches of the river by the unwanted sludge discharged through the sluices. Wishing to concentrate attention to the problem of effecting its separation from the diverted canal water is the reason for not making the matter of sludge disposal a feature of the paper. However, he is glad to see the point raised in the discussion.

In the river bed immediately down stream from the diversion structure, deposition of the surcharge of soil above the capacity soil load of the diminished flow may be expected to produce unstabilized gradients and may cause some expenditure for removing any adverse condition against the down-stream toe of the Imperial Dam, resulting from this deposition.

If the sluicing water and by-passed flow for other users of established water rights, are limited to minimum requirements, the usual stream to be expected would be reduced from its former river magnitudes that were laden with silts and sands originating in and above the canyon section of the watershed, to the comparatively small volumes of flow sufficient for a large-sized canal, and carrying a capacity soil load which originates in the 250-mile length below Boulder Dam. The aggregate available area contributing to this soil load, has been stated as about 25 000 acres, from which on an assumed 10-year period for removal, an averaged annual volume of 31 250 acre-ft is estimated; such a quantity is relatively insignificant when compared with the present vast soil volumes annually lodged on the lower delta. Therefore, there should be little apprehension there, as to the future soil quantities that may be expected, during the life of the Boulder Reservoir; and if the flow is occasionally increased to quantities above these minimum requirements, the ultimate soil volumes to be removed are not changed, and the sluicing soils may be insufficient to contribute capacity soil loads, whence varying degrees of turbidity and bed load are quite as likely in the lower river regions in the future with the lesser stream flows.

The failure in 1931 of a deep steel sheet-piling sea-wall, retaining a fill dredged from the estuary of the Los Angeles River produced investigations in which it was essential to know the correct unit weight of the fill. A mechanical analysis showed fine silts to predominate, 58% passing a No. 200 mesh sieve. The weight of the soil was 120 lb per cu ft, with its water content, as removed from the fill. Engineers and textbooks expressed variant viewpoints as to the unit to be used in calculating the fill pressure. Some assumed that the water contained in the fill (the joints of the sheet-piling not being water-tight) counteracted the ocean hydrostatic pressure of 65 lb per cu ft, using the difference of 56 lb per cu ft as the effective unit loading produced by the fill. This would be an acceptable assumption provided the soil particles were spherical or granular, when the interstices or voids would, of course, be interconnected; but with the lamina-shaped particles coming to rest in still water with their flat sides horizontal, the structuralization of the resulting soil-fill entraps the water in the voids which probably can not be interconnected, such water being an integral part of the fill, and the laboratory weight of 120 lb is that which should be used. Thus, the safety of important structures is sometimes dependent on the weighing of all the physical characteristics of the soil particles, the usual considerations of mechanical analysis and the percentage of voids being insufficient if taken without thought of the structuralization of the particles.

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TRANSACTIONS

Paper No. 1869

MODEL LAW FOR MOTION OF SALT WATER THROUGH FRESH

By Morrough P. O'Brien[1] and John Cherno,[2]
Associate Members Am. Soc. C. E.

With Discussion by Messrs. W. E. Howland, Herbert D. Vogel, C. E. Grunsky, John B. Drisko, G. H. Keulegan, and Morrough P. O'Brien and John Cherno.

Synopsis

In connection with the investigations of the proposed "Salt-Water Barrier" in San Francisco Bay by the United States Engineer Department, certain questions were raised as to the action of salt water when flowing through fresh water. This problem was studied by means of hydraulic models, in the use of which the dimensional relations between model and prototype for similar flow were of primary importance.

Investigation showed that a model, geometrically similar to its prototype, could not be used for a study of the relative movement of salt water and fresh water, but that it must be distorted so that the scales for vertical dimensions, horizontal dimensions, and salinity satisfy a definite relation, which is expressed by Equations (14) of the paper.

Introduction

In connection with an investigation of the "salt-water barrier," that has been proposed as a means of keeping sea water from advancing into the delta of the Sacramento and San Joaquin Rivers, in California, certain problems arose regarding the motion of salt water when placed in contact with fresh water. All the sites proposed for the barrier would require locks for vessels of deep draft operating under an average difference in elevation in favor of fresh water of about 3 ft., with a depth over the sill of 40 ft. for the larger locks. At the Dillon Point site, there would be an adverse salt-water head

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Note.—Published in December, 1932, *Proceedings*.

[1] Assoc. Prof., Mech. Eng., Univ. of California, Berkeley, Calif.

[2] Sacramento, Calif.

during part of the higher tides. These conditions made it possible for large volumes of salt water to pass through the locks into the fresh-water pool with each inbound lockage and thus defeat the purpose of the barrier. As the available data were scanty and indefinite, and to some extent conflicting, it was decided to study the problem through the use of hydraulic models, and, as the first step, a series of experiments were made in the Hydraulic Laboratory of the University of California to determine the model law governing the motion of salt water through fresh water. Although considerable information was obtained regarding dilution and methods of flushing salt water out of fresh-water reservoirs, this paper is concerned only with the derivation and experimental confirmation of the model law governing the dynamical aspects of the problem.

NOTATION

The notation used in this paper is adapted to the suggestions of the Society's Special Committee on Irrigation Hydraulics.[8] Symbols that are used only in derivations and do not appear in the final formulas, will be introduced at their proper place and are omitted from the following list:

b = scale ratio.
d = depth of water, in feet; d_h = gate opening.
d' = depth in model; $b_d = \frac{d'}{d}$ = scale ratio for vertical dimensions.
g = acceleration of gravity.
l = width of lock.
$s = G_1 - G_2$ = salinity.
s' = salinity in model; $b_s = \frac{s'}{s}$ = scale ratio for salinity.
t = time, in seconds.
w = 62.5 lb. per cu. ft. = unit weight of water.
A = total area covered by salt water at any time, measured in terms of $V_0 \, l$.
G = specific gravity; G_1, of heavy liquid; G_2, of light liquid.
K = model characteristic $= \frac{L_0}{d^{2.5}\, s^{0.5}}$.
L = longitudinal distance along the channel; L_0 = length of lock.
L' = length in model; $b_L = \frac{L'}{L}$ = scale ratio for horizontal dimensions.
V = velocity of salt-water wave front, in feet per second; V_0 = initial velocity of salt-water wave front, in feet per second.
ν = kinetic viscosity as defined on page 585.

DESCRIPTION OF PHENOMENON

Before developing the theory or discussing the experimental results, a brief description of the phenomenon observed will be useful in defining the problem. The channel shown in Fig. 1 was arranged so that a vertical gate divided it into two compartments. The smaller compartment was filled with colored salt water while the remainder of the channel was filled with fresh water.

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[8] *Proceedings*, Am. Soc. C. E., May 1932, p. 729.

The water surfaces on the two sides of the gate were brought to the same elevation, and the gate was then removed suddenly. The heavier salt water slid under the fresh water and moved along the bottom of the channel with a peculiar tumbling motion while the fresh water moved in the opposite direction along the surface.

The condition immediately after the gate was removed is shown in Fig. 2, the white streak in the middle of the photograph marking the original position of the gate. Fig. 3 shows the advancing wave front, which for sea water (specific gravity = 1.026), with a depth of 1.2 ft., had a velocity of approximately 0.50 ft. per sec. At some instant subsequent to the removal of the gate, the fresh water, moving into the smaller chamber, reached the rear wall, and the salt water continued to move along the channel as a slug of constantly diminishing height and increasing length. The initial height of the salt-water wave was almost exactly one-half the total depth as appears in Fig. 2. In Fig. 3, showing the wave at eight lock lengths from the initial position, the height has decreased to four-tenths of the depth. In Fig. 4, the advancing wave has piled up against the end of the channel and a reflected wave is about to start back along the surface of separation of the two liquids. Fig. 5 is a view of an experimental channel at Turlock, Calif., discussed subsequently.

THEORY

Consider a channel of rectangular cross-section and horizontal bottom, divided into two compartments containing liquids of different specific gravities. It is evident that, when the two liquid surfaces are at the same elevation, the forces on the two sides of the partition are not equal and that a relative motion of the two liquids will result when the partition is removed. Referring to Fig. 6, the line, *A O B,* represents the surface of separation of the two liquids at some time subsequent to an instantaneous removal of the gate. The heavier liquid is moving along the bottom, displacing a part of the lighter liquid, which is moving in the opposite direction along the surface.

Observation shows no motion of the salt water to the left of Section *C-C* and of the fresh water to the right of Section *E-E,* but all parts of both liquids acquire their full velocities, when they are reached by the planes, *C-C* and *E-E,* in such a short time that, for all practical purposes, infinite acceleration can be assumed. It was further observed that the point, *O,* is at one-half the depth and that no change in surface elevation occurs, so that the quantities of fresh water and salt water passing the gate section are equal and move with equal velocity, $V_0 = \frac{dx}{dt}$. The unbalanced force acting toward the right on the mass, $M = 2\,x\,\frac{d}{2}\,G_1\,\frac{w}{g} + 2\,x\,\frac{d}{2}\,G_2\,\frac{w}{g}$, of water between Sections *C-C* and *E-E* is;

$$w\,G_1\,\frac{d^2}{2} - w\,G_2\,\frac{d^2}{2} = w\,s\,\frac{d^2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and is equal to the change of the momentum, $M\,V_0$, per second. Therefore,

$$w\,s\,\frac{d^2}{2} = \frac{d}{dt}\,(M\,V_0) = \frac{w}{g}\,d\,(G_1 + G_2)\,\frac{d}{dt}\left(x\,\frac{dx}{dt}\right) \quad \ldots\ldots\ldots (2)$$

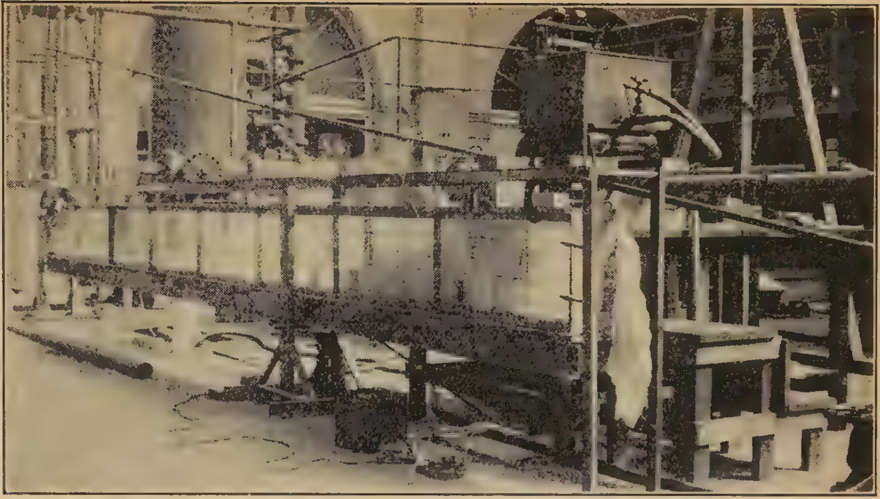

FIG. 1.—VIEW OF EXPERIMENTAL CHANNEL WITH GLASS SIDES.

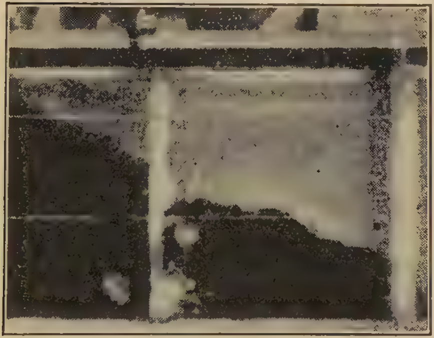

FIG. 2.—SALT WAVE LEAVING LOCK.

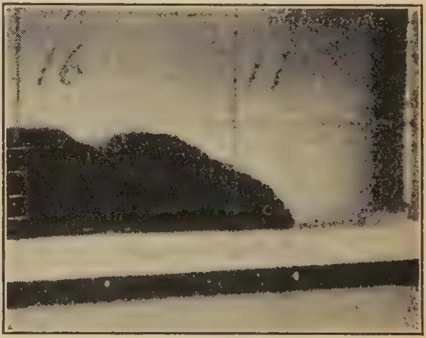

FIG. 3.—SALT WAVE AT ABOUT EIGHT LOCK LENGTHS FROM GATE.

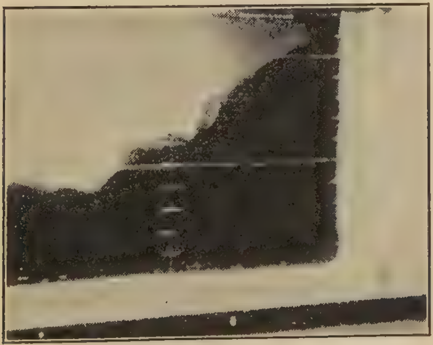

FIG. 4.—SALT WAVE STRIKING BULKHEAD END OF CHANNEL.

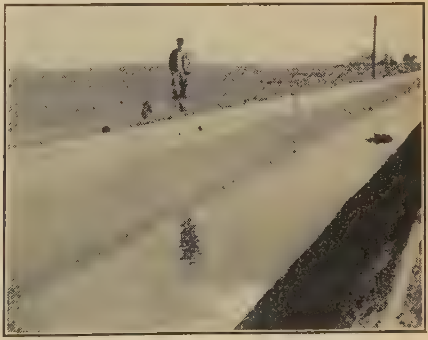

FIG. 5.—EXPERIMENTAL CHANNEL AT TURLOCK, CALIF.

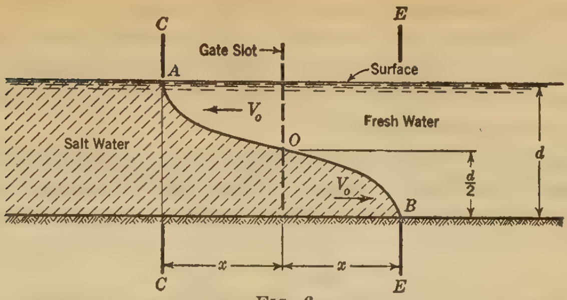

FIG. 6.

The first integral, considering that $x = 0$ for the moment, $t = 0$, when the gate is removed, is,

$$s\,\frac{d}{2}\,t = \frac{G_1 + G_2}{g}\,x\,\frac{dx}{dt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The second integral is, therefore,

$$x = \pm\, t\sqrt{\frac{g\,d\,s}{2\,(G_1 + G_2)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Therefore,

$$V_0 = \frac{dx}{dt} = \pm\sqrt{\frac{g\,d\,s}{2\,(G_1 + G_2)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Since V_0 is independent of x and t, it follows that all salt water and fresh water is brought instantly to an initial velocity, V_0, until either Section *C-C* or Section *E-E* reaches the end of the testing flume. After that the foregoing assumed conditions no longer exist, and the subsequent advance of the salt water takes place, as shown in Fig. 7.

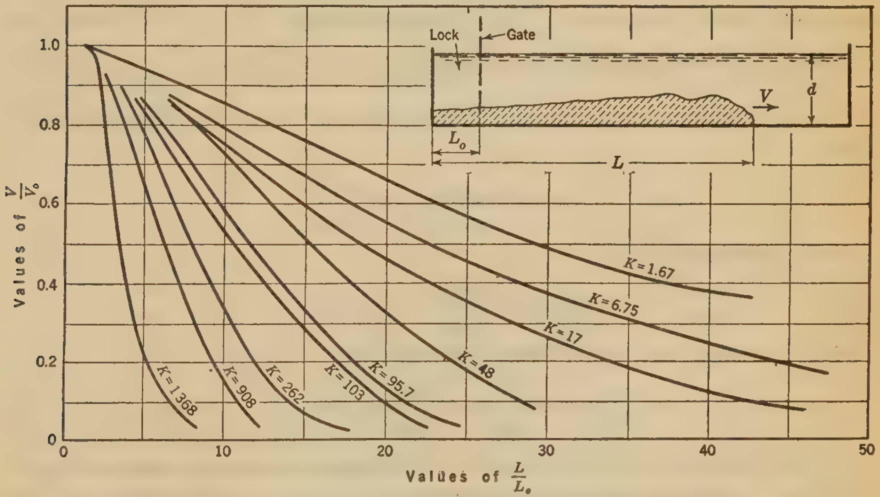

FIG. 7.—RELATION BETWEEN RATE OF DECREASE OF VELOCITY AND MODEL CHARACTERISTIC, *K* (LABORATORY EXPERIMENTS).

Tests might be expected to show initial velocities slightly less than V_0 as computed by Equation (5) because no friction losses have been taken into account. However, it will be shown presently that delays in opening the gate will cause initial velocities larger than V_0, so that no prediction of a systematic deviation can be made.

Another method of obtaining Equation (5) is to consider the potential energy converted into kinetic energy during the downward motion of the heavier liquid. If, in Fig. 8, d_g is the vertical distance from the sill of the gate to the center of gravity of a volume of liquid, B, which ultimately passes through the gate, the velocity of efflux can be obtained on the assumption that a counter flow of the lighter liquid occurs with the surface of separation at $\frac{d_h}{2}$, and with a velocity equal to the outflow velocity of the salt water. This is equivalent to assuming that the surfaces on the two sides of the gate remain at the same elevation after the gate is opened, and is based on direct observation. The center of gravity of B falls through the distance, $d_g - \frac{d_h}{4}$, but an equal volume of the lighter liquid is raised through the same distance. The decrease in the potential energy of the system is, then,

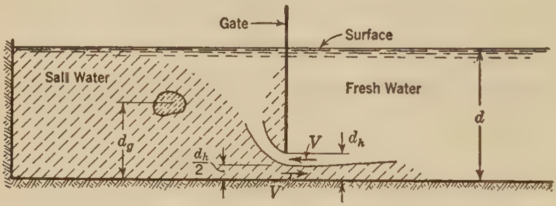

FIG. 8.

$$E_d = B\,w\,s\left(d_g - \frac{d_h}{4}\right)$$

The kinetic energy is,

$$E_K = \frac{B\,w\,V_1^2}{2\,g}\,(G_1 + G_2)$$

and, if the slight frictional losses are neglected,

$$\left(d_g - \frac{d_h}{4}\right)s = \frac{V_1^2}{2\,g}\,(G_1 + G_2) \;\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

If the lock is filled with salt water and the gate is opened a small distance, the initial velocity is obtained by making $d_g = d$ and $d_h = 0$, since the first fresh-water particles passing through the gate rise from the bottom to the surface. The initial velocity of efflux is, then,

$$V_1 = \sqrt{\frac{2\,g\,d\,s}{G_1 + G_2}} = 2\,V_0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Equation (7) reduces to the ordinary orifice equation when $G_2 = 0$, corresponding to the discharge of water into air. If the gate is removed, $d_g = \frac{d}{2}$, $d_h = d$, and V_1 becomes equal to V_0 as obtained from Equation (5) for the same conditions (that is, removal of the gate).

Since a finite time is required for raising the gate, it follows that the observed initial velocity will lie between the value given by Equations (5) and (7), approaching the former value the more closely, the shorter the time of opening.

Referring to Fig. 6, the surface particles of the lighter liquid are moving toward the left and the question arises as to whether their velocity is created by a drop, $\frac{V_0^2}{2g}$, of the surface above Point B, as Bernoulli's theorem would require for steady flow. For $G_d = 0.025$ and $d = 1$, the drop, $\frac{V_0^2}{2g}$, would be about 0.003 ft. and, therefore, visible. The only surface disturbances observed, however, were those directly attributable to the opening of the gate and these were much smaller than the aforementioned amount.

As the total volume of liquid in the testing flume is constant, a drop of the surface at any place would have to be made up by a rise of the surface at some other place; that is, water would have to be lifted from below the original surface to above it, and, therefore, no drop of the surface could furnish the energy required to set water in motion. On the contrary, it would diminish the kinetic energy. Although it is not the intention to investigate herein the mechanism by which the different parts of the two liquids are set in motion, it can be stated that vertical displacements of the surface are not only unnecessary for the creation of the observed movements, but are opposed to them. It follows from the derivation of Equation (6) that such displacements would be reflected in a deficiency of V_0 and as shown subsequently, if such a deficiency exists at all, it is negligible.

THE MODEL LAW

The term, "model," refers to any hydraulic system by means of which the behavior of another system called the prototype, can be predicted, and the term, "model law," refers to the method of such prediction. The model law is a set of rules which assigns to each point of the prototype a corresponding point in the model and assigns the relations existing between the physical properties at corresponding points.

In the usual case, the required point-to-point relationship between the model and its prototype is obtained by making them geometrically similar at least to the extent of using only one scale ratio for horizontal dimensions and one for vertical dimensions. When these scale ratios are equal, the model and prototype are geometrically similar in the ordinary sense; otherwise, the model is "distorted." Another requirement is that the ratio of the measured quantities at pairs of corresponding points shall be the same for all points. For example, if the velocity in the model is one-half the velocity in the prototype at any pair of corresponding points, the velocity at any point in the model should be one-half the velocity at the corresponding point in the prototype. In other words, the scale ratio of velocities is to be a constant and a similar relation should hold true of all other measured quantities.

Whether it is possible to build models that represent their prototypes in every detail will not be discussed here. In practice, models are always used for the study of a limited number of phenomena and should be built so that these phenomena are represented properly. Minor associated phenomena must often be represented incompletely or even incorrectly and the success of a model experiment depends upon whether these neglected phenomena are controlling factors.

The present problem is the derivation of the model law governing the intrusion of salt water into fresh water in a shallow basin such as San Francisco Bay. The ultimate problem is a study of the distance traversed by the salt water, the dilution it undergoes during the motion, and its velocity at points along its path. It is known from direct observation that the vertical motion involved can be neglected because it is small as compared with the horizontal motion and because it occurs in a time that is short as compared with the time required for the entire process, and the model law will be based upon these assumptions.

Transference of the data from the model to the prototype will require the six ratios: Horizontal dimensions; vertical dimensions; velocities; time; salinity; and dilution. The dilution itself is a ratio and has no dimensions and, consequently, its value should be the same in model and prototype at corresponding points. The first five scale ratios are not independent and it is necessary to determine the relation between them.

In the prototype, the velocity is, $V = \dfrac{dx}{dt}$, and the corresponding quantity in the model is, $V' = \dfrac{dx'}{dt'} = \dfrac{b_L}{b_t}\dfrac{dx}{dt} = \dfrac{b_L}{b_t} V$. The velocity ratio is, therefore,

$$\frac{V'}{V} = b_v = \frac{b_L}{b_t} \quad \text{..........................(8)}$$

From Equation (5), the initial velocities are,

$$V_0 = \sqrt{ds}\,\sqrt{\frac{g}{2\,(G_1 + G_2)}}$$

in the prototype, and,

$$V'_0 = \sqrt{d'\,s'}\,\sqrt{\frac{g}{2\,(G'_1 + G'_2)}}$$

in the model. Therefore,

$$\left(\frac{V'_0}{V_0}\right)^2 = \frac{d'}{d} \times \frac{s'}{s} \times \frac{G_1 + G_2}{G'_1 + G'_2}$$

or,

$$b^2_v = b_d\,b_s\,b^{-1}_{(G_1 + G_2)} \quad \text{..........................(9)}$$

If salt water and fresh water are to be used in the model, $G_2 = 1.00$, and G_1 ranges from 1.00 to 1.20, so that $b_{(G_1 + G_2)}$ ranges from about 0.9 to 1.1 and ordinarily will be close to 1.0. For simplicity this ratio will be assumed equal to 1.00 in the following discussion unless otherwise specified. On this basis, Equation (9) becomes,

$$b^2_v = b_d \, b_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

A third relation between the scale ratios is found in the general equations of internal friction, which have the form,

$$\frac{w}{g}\frac{dV}{dt} + \frac{dp}{dx} - \mu \left\{ \frac{d^2V}{dx^2} + \frac{d^2V}{dy^2} + \frac{d^2V}{dz^2} \right\} = 0$$

for the X-direction, which is horizontal. The first term represents the inertia of a particle; the second, the unbalanced force acting on it in the X-direction; and, the third, the internal friction. Here, μ is a coefficient of internal friction which is ordinarily the coefficient of absolute viscosity. In the present case, μ may differ from the viscosity of either of the liquids in contact. The absolute viscosity divided by the density, $\frac{w}{g}$, is usually referred to as the kinematic viscosity and will be represented by ν. It will be assumed to have the same value for both model and prototype unless otherwise specified.

A similar equation can be written for the Y-direction which is also horizontal. The Z-direction is not considered since vertical motions are neglected.

Let P denote a hydraulic force. Fig. 9 shows that $\frac{dp}{dx}$ may be written as $\frac{d\,(w\,H\,s)}{dx}$ because of the horizontal water surface. Direct observation shows that salt water tends to move along the bottom of a channel as a coherent slug and that fresh water moves over the top of it at about the same velocity in the opposite direction. The velocity gradient, therefore, is large at the surface of separation and relatively small in all other directions and at all other positions. In other words, practically all the work of internal friction is done in the transition layer.

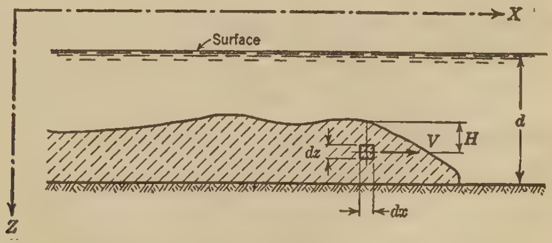

FIG. 9.

In view of the foregoing considerations, the equation of motion is simplified to,

$$\frac{dV}{dt} + g\,\frac{d\,(H\,s)}{dx} - \nu\,\frac{d^2 V}{d\,z^2} = 0 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

after dividing through by $\frac{w}{g}$. Writing the corresponding equation for the model and remembering that H is a vertical dimension,

$$\frac{d\,V'}{dt'} + g\,\frac{d\,(H'\,s')}{dx'} - \nu\,\frac{d^2\,V'}{d\,z'^{\,2}} = \frac{b_v}{b_t}\,\frac{d\,V}{dt} + \frac{b_d\,b_s}{b_L}\,\frac{d\,(H\,s)}{dx} - \frac{b_v}{b^2_d}\,\nu\,\frac{d^2\,V}{dz^2} = 0 \;..(12)$$

Equations (11) and (12) can be equal only if,

$$\frac{b_v}{b_t} = \frac{b_d\;b_s}{b_L} = \frac{b_v}{b^2_d} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The physical meaning of Equation (13) is that the inertia, pressure, and friction forces in the prototype must be reduced in the same ratio in the model.

Equations (8), (10), and (13) are to be satisfied simultaneously. Taking b_d and b_s as the independent variables, these equations give as the model relations:

$$b_L = b_d^{2.5}\,b_s^{0.5} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14a)$$

$$b_v = b_d^{0.5}\,b_s^{0.5} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14b)$$

$$b_t = b_d^2 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14c)$$

The ratios, b_s and b_d, have been chosen as the independent variables because they are easiest to control in practice. Equation (14*a*) is of primary importance in the construction of a model. Replacing the ratios by characteristic dimensions such as the length of a lock, L_0, the depth, d, and the initial salinity, s, Equation (14*a*) gives,

$$\frac{b_L}{b_d^{2.5}\,b_s^{0.5}} = 1 = \frac{L'_0}{L_0}\left(\frac{s}{s'}\right)^{0.5}\left(\frac{d}{d'}\right)^{2.5}$$

Separating the quantities for model and prototype,

$$\frac{L'_0}{d'^{\,2.5}\,s'^{\,0.5}} = \frac{L_0}{d^{2.5}\,s^{0.5}} = K \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which, K is the simplified criterion of similarity.

Retaining both g and ν as variables, the complete model law is,

$$\frac{L_0\,\nu}{g^{0.5}\,s^{0.5}\,d^{2.5}} = K' \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

which is dimensionless. Equation (15) is applicable to the case of salt water and fresh water because ν is approximately constant, but Equation (16) may be necessary if liquids are used in the model entirely different from those in the prototype. The ratio, $b_{(G_1+G_2)}$, might also have to be included in this case.

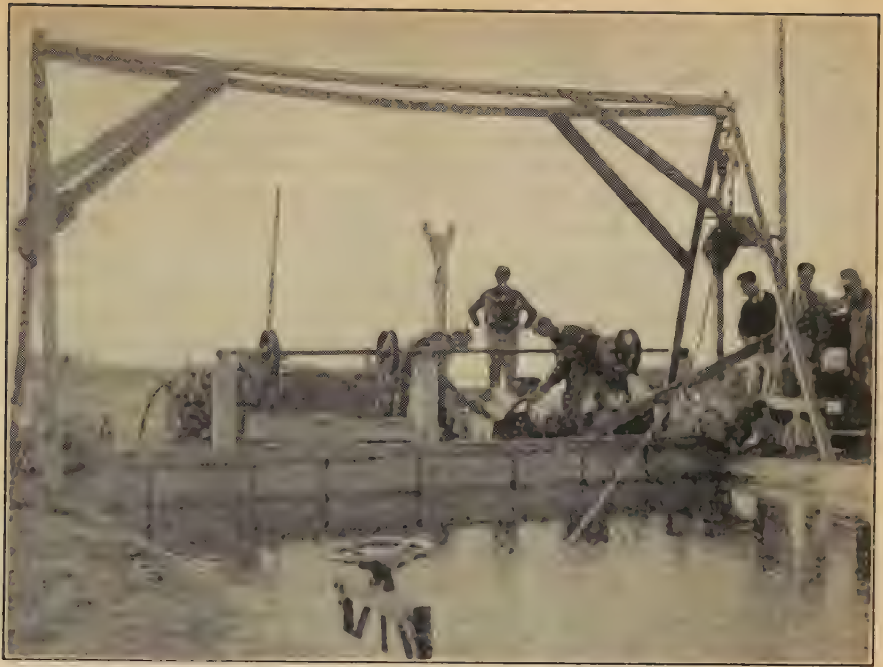

FIG. 10.—TURLOCK EXPERIMENTS: GATE LOWERED FOR START OF EXPERIMENT

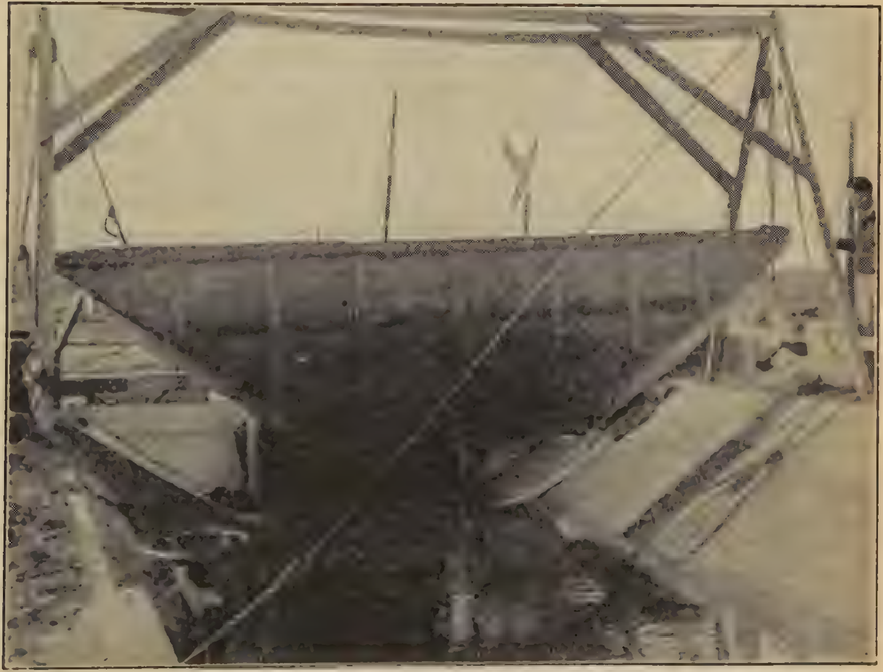

FIG. 11.—TURLOCK EXPERIMENTS: GATE RAISED.

EXPERIMENTAL PROCEDURE

The glass-sided channel (Fig. 1) used in the first series of experiments, was 24.4 ft. long, 0.5 ft. wide, and 1.25 ft. deep, placed so that the bottom was horizontal. A thin, sheet-metal gate, moving in vertical guides placed above the channel proper and sealed with rubber strips, coated with grease, was used instead of a lock-gate. The volume displaced by the gate was insignificant, and the surface disturbances resulting from its removal were not sufficient to have an appreciable effect on the velocities. The specific gravities of the liquids were measured by a hydrometer or a specific gravity balance, and time intervals by a chronograph and stop-watches. A co-ordinate system in feet and tenths was drawn on the glass plates with a grease pencil.

The liquid used to fill the lock was a solution of crude granulated rock salt in water, colored to a deep red with potassium permanganate. As the gate was unable to withstand any considerable side thrust, the channel was first filled to the desired depth with fresh water, and the gate was then put in position and sealed with cup grease. The fresh water was siphoned from the surface of the lock chamber as the saline solution was discharged at the bottom, and the two water surfaces were brought to the same elevation by point-gauges reading to 0.001 ft. The lock chamber was then thoroughly stirred by bubbling before the specific gravity was measured. The behavior of the salt water following the removal of the gate has been described previously.

Following the laboratory experiments, a few additional tests were made in a concrete-lined irrigation canal of trapezoidal section at Turlock, Calif. (See Fig. 5.) This channel had a bottom width of 6 ft., side slopes of 1 on $1\frac{1}{2}$, a maximum depth of 4 ft., a bottom slope of 1 on 2 000, and a length of 2 600 ft. The gate, which consisted of a trapezoidal wooden frame covered with oil-cloth, was placed at the lower end of the experimental section so that the advancing wave traveled up along the bottom of the channel. The manner of conducting the tests was similar to that in the laboratory, but the probable error was much greater. In addition, a current along the channel, due to leakage past the gates, was noticeable in some experiments. During certain experiments, wind-driven surface currents created counter currents on the bottom opposing the progress of the salt solution. The gate was lifted by block and tackle and the time required to lift it clear of the water surface was relatively greater than in the laboratory, a condition which, according to Equation (7), would result in a relatively higher initial velocity. The channel and gate used at Turlock appear in Figs. 10 and 11.

AGREEMENT BETWEEN THEORY AND EXPERIMENT

In Table 1, the measured initial velocities in the laboratory experiments are compared with the velocities computed from Equation (5). The data were picked so as to give as a wide range of d, s, and L_0 as possible and to include tests giving the maximum deviation from the formula. The differences between the experimental results and the equation are erratic and do not indicate any general tendency for the actual velocity to be greater or less than that given by the formula. Considering the data of Table 1, it can be stated that Equation (5) represents the initial velocity in a rectangular channel with a probable error of 2 per cent.

TABLE 1.—INITIAL VELOCITIES MEASURED IN LABORATORY

Test No.	d, in feet	L_0, in feet	s	V_0, in feet per second (computed)	V_0, in feet per second (observed)	Percentage, deviation
A-2......	1.20	1.79	0.029	0.528	0.595	+11.2
A-5.......	0.80	1.79	0.030	0.437	0.416	—4.8
A-6......	0.40	1.79	0.032	0.319	0.298	—6.3
A-25.....	0.415	0.90	0.0302	0.316	0.297	—6.0
A-23.....	0.412	0.32	0.0319	0.324	0.298	—8.0
A-14......	1.20	8.95	0.032	0.554	0.532	—4.0
A-10.....	1.20	0.39	0.023	0.470	0.436	—7.1
A-4......	1.20	1.79	0.015	0.380	0.379	—0.2
A-12......	1.19	1.79	0.063	0.767	0.912	+18.9
A-13......	1.20	1.79	0.101	0.970	0.995	+ 2.6

Table 2 shows the initial velocities measured in the irrigation channel at Turlock. As would be expected from the time required to open the gate, the observed velocities are in excess of the computed values, and the percentage deviation increases with the depth. Segregating the tests according to the depth, the average deviations for depths of 2, 3, and 4 ft. are found to be 4.0%, 12.6%, and 19.6%, respectively.

TABLE 2.—INITIAL VELOCITIES MEASURED IN TURLOCK, CALIFORNIA

Test No.	d_0	L_0	G_0	V_0 (Computed)	V_0 (Observed)	Error
T-1.........	2.1	10.5	0.0091	0.392	0.374	-4.6
T-2.........	3.1	10.5	0.0110	0.523	0.578	10.1
T-3.........	3.95	10.5	0.0113	0.598	0.693	15.9
T-4.........	3.85	20.6	0.0110	0.582	0.735	26.2
T-5.........	1.8	20.2	0.0123	0.421	9.445	5.7
T-6.........	3.0	20.3	0.0100	0.490	0.540	10.2
T-7.........	3.8	5.4	0.0083	0.503	0.647	28.5
T-8.........	2.9	5.2	0.0095	0.470	0.536	14.0
T-9.........	2.25	5.7	0.0094	0.411	0.459	10.8
T-10........	3.05	6.4	0.0234	0.752	0.873	16.2
T-11........	4.00	6.4	0.0215	0.828	0.893	7.9

SPREADING EXPERIMENTS

An additional check on the model law was made in a sheet-metal tank of irregular shape, approximately 25 ft. long, 6 ft. wide, and 1 ft. deep. The method of conducting the experiments was entirely similar to that in the glass channel with the difference that the bottom was covered with a thin layer of sand, on which a co-ordinate system of white lines was drawn with plaster of Paris. A lock, 1.04 ft. long and 0.82 ft. wide, with wing-walls, forming a continuation of the sides of the lock and projecting 0.4 ft., was placed at the side of the tank for part of the tests and at the end for the remainder. This arrangement permitted lateral motion of the salt solution which had not been considered in any of the previous experiments. In deriving Equation (12) only the forces in the *X*-direction were considered; a similar line of reasoning applied in the Y-direction would yield the same result, and, consequently, the same model characteristic, K, should completely describe the motion in all systems having horizontal dimensions proportional to L_0 and depth proportional to d.

The results of these experiments on the effect of the spreading are shown in Fig. 12, for the lock at the end of the tank (Fig. 12(*a*)), and the side, (Fig. 12(*b*)), respectively. The symbols used have the following significance:

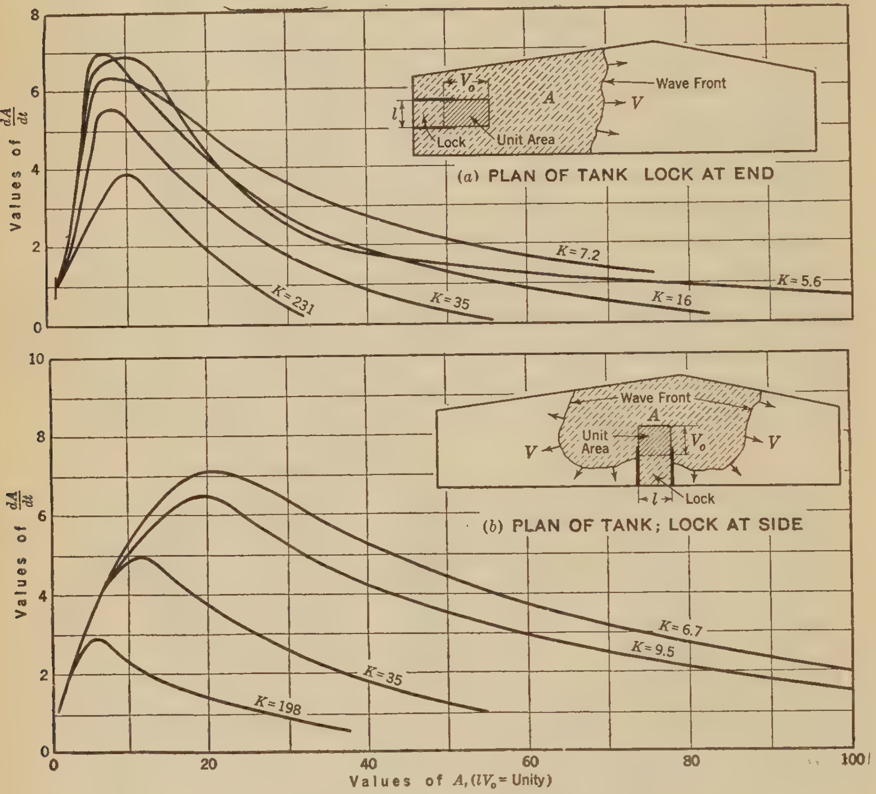

FIG. 12.—RATE OF SPREADING SALT WATER OVER A HORIZONTAL PLANE.

$$V_0 = \sqrt{\frac{gd}{2}}\sqrt{\frac{s}{G_1 + G_2}} = \text{the initial velocity of the salt water.}$$

l = the width of the lock.

A = the total area covered by salt water at any time, measured in terms of $l\,V_0$.

$\frac{dA}{dt}$ = the time rate of increase of A.

As explained previously, the bottom of the channel was covered with a co-ordinate system of white lines, and as the wave spread out from the lock in all directions, the time at which the wave front reached various points in that system was noted. Contours representing the area covered at any time were then drawn and, by measuring the areas inside these lines, the values of A and $\frac{dA}{dt}$ were obtained.

In the initial stage of the motion, the velocity is relatively great and the time is difficult to measure precisely, so that the experimental points show considerable scattering in this region; but as the velocity decreases, the points fall more regularly. All the curves exhibit the same general shape and, with only one exception, they arrange themselves according to the value of K. The length of the lock was constant for all these experiments, but the depth ranged from 0.18 to 0.8 ft. and the specific gravity of the salt solution ranged from 1.046 to 1.098.

The model law derived previously indicated that when the value of $\frac{L_0}{d^{2.5}\ s^{0.5}}$ is the same for two structures having horizontal dimensions proportional to L_0 and vertical dimensions proportional to d, the entire motion of the salt water should be similar. A few of the tests made in the glass channel are shown in Fig. 7. The ordinates represent the velocity in percentage of the initial velocity of advance. The abscissas represent the number of lock lengths from the closed end of the lock to the point at which the velocity was measured. It is to be noted particularly that all the curves have the same general shape and that they distribute themselves according to the value of K. Twenty-three curves similar to those in Fig. 7 were used to obtain Fig. 13, which shows the relation between the number of lock lengths from the closed end of the lock, at which the velocity is reduced to a certain percentage of the initial velocity, and K, the model characteristic.

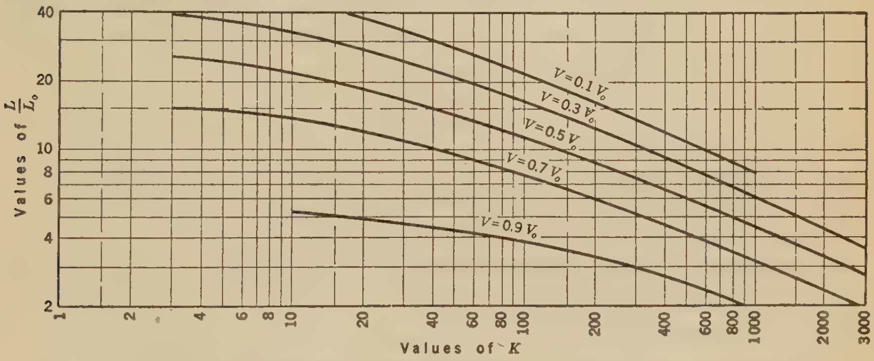

FIG. 13.—RELATION BETWEEN RATE OF DECREASE AND MODEL CHARACTERISTIC, K.

The curves of Fig. 13 permit drawing the velocity curve for any value of K. The actual velocity in any particular case may be obtained by multiplying each ordinate by the initial velocity given by Equation (5).

Fig. 14 shows the velocity curves obtained at Turlock. As mentioned previously, the initial velocity in most cases is high, but the curves for all twelve tests arranged themselves according to the value of K. Only six of the curves are shown on the graph, in order to avoid confusion, but the remainder showed equal regularity both in shape and location.

In Fig. 14, curves (dotted) drawn from the data in Fig. 13 are compared with the actual data obtained at Turlock for equal values of *K*, the model characteristic. The initial velocity at Turlock was greater than that observed in the laboratory, but the velocity diminished more rapidly, due principally to the upward slope of the bottom. Other contributing factors in the more rapid decrease in velocity at Turlock were the currents of fresh-water leakage through the upper bulkhead, and the wind. Allowing for all sources of error in these large-scale tests and for the fact that the trapezoidal channel is not quite equivalent to the rectangular channel used in the laboratory, the writers consider the results of the Turlock tests to be a satisfactory confirmation of the results obtained in the laboratory.

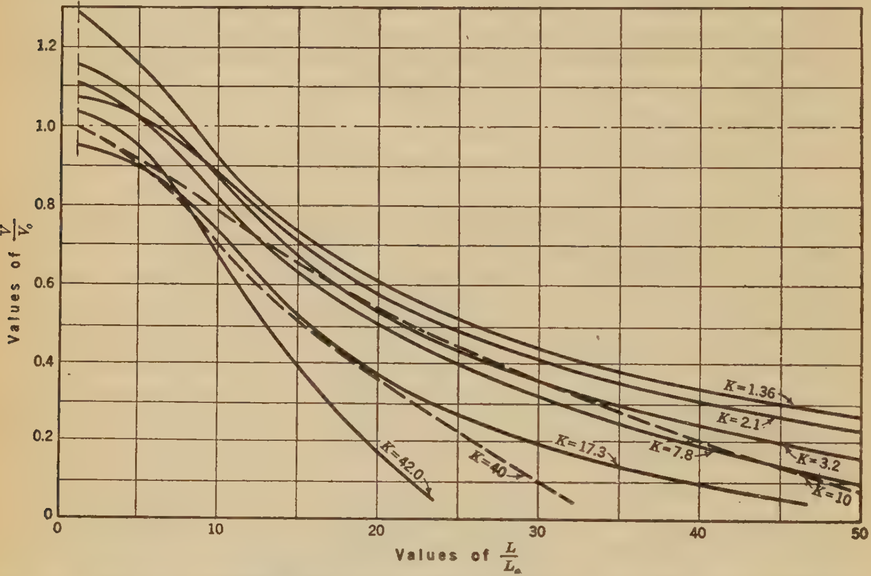

FIG. 14.—RELATION BETWEEN RATE OF DECREASE OF VELOCITY AND MODEL CHARACTERISTIC, *K*; TURLOCK EXPERIMENTS.

CONCLUSIONS

From an assumed character of the forces involved in the motion of salt water through fresh water, the model law governing the similarity was determined to be $\frac{L_0}{d^{2.5}\ s^{0.5}} = \text{constant} = K$, in which, L_0 is any characteristic horizontal dimension, *d* is a characteristic vertical dimension, and *s* is a characteristic salinity.

The experimental results show that, regardless of the absolute values involved, the velocity curves distribute themselves according to the values of *K*, and for two tests having values of *K* that are approximately equal, the curves lie very nearly parallel throughout their length. From this, it may be concluded that, if the difference in the values of K for two tests is zero, the velocity curves will coincide, and, consequently, K is the criterion for dynamical similarity.

Furthermore, since the model law, as derived theoretically, coincides with that found experimentally, the fundamental assumption made in the derivation of the law — that is, the assumption that the internal friction acts like a force equal to $\nu \frac{d^2 V}{dZ^2} dx\, dy\, dz$ — is essentially correct and no forces other than the two considered in deriving the model law are of major importance.

One other force to which attention was paid is the friction at the bottom of the testing flume. Most of the tests were run on a hardwood bottom painted with red lead. Similar tests were run with the bottom covered with coarse sand for comparison, but no difference in the velocity of the salt water was found. Since the fresh water moves in a direction opposite to that of the salt water in the upper part of the flume, the vertical velocity gradient is exceedingly large at the surface of separation of the two liquids, and it is natural that the friction developed in these transition layers should be great enough to govern the motion almost completely.

It is to be emphasized that Equation (15) is valid only for cases in which the principal motion is horizontal. It is also restricted to experiments in which the kinematic viscosity is the same in both model and prototype; otherwise, Equation (16) is to be used. Another restriction is that the velocities and depths in the model must not produce laminar flow, if turbulent flow exists in the prototype.

The present experiments do not indicate any quantitative limitations of the validity of Equation (15). However, the writers are of the opinion that an actual lock having a length twenty to thirty times its depth could be studied in a model having a length only one-half, or even one-third, of the depth using salt water in the model, but they do not feel that the present experiments alone justify the use of such a combination as ether and mercury to represent the fresh and salt water, respectively. Further experimental work should be done before such a drastic step is taken.

As an illustration of the application of these conclusions, assume that model tests are to be made concerning the distribution and dilution of salt water flowing from the Ballard Locks into the Lake Washington Ship Canal at Seattle, Wash., and compute the dimensions to be used in the construction of a model according to the model law derived in this paper. The actual dimensions are: Length of canal from locks to Lake Washington, 33 000 ft.; depth, length, and width of the locks, 37, 825, and 80 ft. respectively. The law gives a relation between three scale ratios, namely, a ratio for vertical dimensions, b_d; one for horizontal dimensions, b_L; and one for salinity, b_s. Two of the scale ratios can be chosen arbitrarily and the third one is then determined by the relation,

$$\frac{b_L}{b_d^{2.5}\ b_s^{0.5}} = 1 \qquad \text{.........................(17)}$$

Table 3 shows the dimensions assumed by the large lock, and by the distance from the locks to the entrance into Lake Washington in models of different scales. A set of vertical scales is chosen, and it is assumed that either the tests would be run with water of the same salinity as that of Puget Sound (which is about 0.02), giving a salinity ratio of unity, or with water of a salinity of 0.2 which is about as high as could be made, and in which case the salinity ratio would be 10:1. The horizontal scales as computed from the model law and the resulting dimensions of the large lock and the distance to Lake Washington in the model, are also shown in Table 3.

TABLE 3.—ILLUSTRATIVE EXAMPLE; DISTRIBUTION AND DILUTION OF SALT WATER INTO LAKE WASHINGTON SHIP CANAL

Vertical scale, b_d	SALINITY SCALE; 1:1					SALINITY SCALE; 10:1				
	Horizontal scale, b_L	Distance to Lake Washington, in feet	Lock Dimensions			Horizontal scale, b_L	Distance to Lake Washington, in feet	Lock Dimensions		
			Depth, in feet	Length, in feet	Width, in feet			Depth, in feet	Length, in feet	Width, in feet
1:2	1:5.6	5 900	18.5	148	14.3	1:1.77	18 700	18.5	466	45
1:5	1:55.9	591	7.4	14.8	1.43	1:17.7	1 870	7.4	46.6	4.5
1:10	1:316	105	3.7	2.6	0.25	1:100	330	3.7	8.25	0.8
1:20	1:1770	18.7	1.85	0.47	0.05	1:560	59	1.85	1.48	0.14

ACKNOWLEDGMENTS

These studies were made by the United States Engineer Department, San Francisco District, in conjunction with the University of California. They formed a part of investigations made for a "Report on Proposed Salt Water Barrier, Sacramento, San Joaquin, and Kern Rivers, California," pursuant to the provisions of House Document No. 308, 69th Congress, First Session.

The investigations were made under the direction of E. H. Ropes, Maj., Corps of Engineers, U. S. A., Assoc. M. Am. Soc. C. E., District Engineer; C. A. Mees, M. Am. Soc. C. E., Principal Engineer, who was in charge of the work with H. G. Gerdes, Assoc. M. Am. Soc. C. E., Associate Engineer as Assistant. Most of the testing was done in the Hydraulic Laboratory of the University of California and the remainder in an irrigation ditch of the Turlock Irrigation District. The writers were in charge of the tests, Professor O'Brien for the University of California and Mr. Cherno for the U. S. Engineer Department.

DISCUSSION

W. E. Howland,[4] Assoc. M. Am. Soc. C. E. (by letter).—This study seems to be a novel and most promising application of the methods of model testing to a baffling and important problem. It would seem to suggest methods for the study of other related problems, such as the spread of sewage over the surface of a salt-water harbor already treated[5] by A. M. Rawn and H. K. Palmer, Members, Am. Soc. C. E., and others.

Referring to the authors' Fig. 6, and considering as a free body that portion of the fluid between a section taken a little to the left of Section *C-C*, and a section taken a little to the right of Section *E-E*, the horizontal forces acting on this free body would be $\frac{1}{2} s w d^2$, toward the right (if the levels are as shown) and also friction forces the resultant of which probably acts toward the left. Are these forces necessarily in equilibrium? Is it not possible that they have a resultant acting to the right that tends to move the entire body in that direction, tending thus to depress the water surface at *C* and raise it at *E*? If this is not so, it is difficult to understand how the fresh water between *C* and *E* can start to move toward the left notwithstanding the authors' statement that the only surface disturbances observed were those directly attributable to the opening of the gate.

Does the interface of the salt and fresh water incline as shown in Fig. 6? If so, one can understand how the pressure gradient in the salt water might be toward the right, imparting motion in that direction. If, however, this interface slopes toward the left, as suggested in Fig. 7, it would seem necessary to assume a considerable decrease of momentum in the direction of motion. Possibly an assumption of mixing that would have the result of clouding the effects, could account for the appearance indicated. If the former assumption is correct, it would seem that the condition illustrated in Fig. 7 would be a secondary one and would not be the condition prevailing at the beginning of the motion. In order to make the salt water continue to move to the right, it would seem that there must be a pressure gradient predominating toward the right within the salt water. Fig. 7 illustrates a most interesting condition, in which all the exterior forces on the moving salt water are tending to slow it down. This might explain the rapid decrease in velocity observed in certain of the experiments.

If the writer's assumptions as to the depression of the water surface near *C* and the corresponding rise near *E*, are correct, it should be possible to detect a movement of the surface water toward *C* in both directions and away from *E* in both directions. Were any such movements detected? Furthermore, it might be supposed that such changes in elevation of surface would give rise to waves of small amplitude which, in time, would cause a jerky or intermittent motion of colored salt water. Was this effect noticed?

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[4] Asst. Prof., Civ. Eng., Purdue Univ., Lafayette, Ind.

[5] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1036.

Messrs. Rawn and Palmer[6] recorded as an observation of their experiments that "colored fresh water moved over the salt water in concentric waves"; but, of course, the conditions in these two experiments are entirely different.

Similar considerations would suggest that when the gate is opened only a slight amount at the bottom, as indicated in Fig. 8, it is possible that the water would move entirely in one direction—that is, toward the fresh water—and that no fresh water would move in the other direction. Therefore, the initial velocity toward the fresh water might be even greater than as computed; that is, it might be nearly as great as $\sqrt{2\,g}\sqrt{\frac{d s}{G_1}}$. Have any direct observation of this case been made?

The writer has obtained the model law (Equation (16)) by methods which he believes to be similar to those used by the authors, by assuming that: (*a*) The impressed forces on a body between Sections *C-C* and *E-E* (Fig. 6) are proportional to $w\ s\ d^2$; (*b*) the resistant forces are proportional to $v d$, x, *or* L, and kinematic viscosity; and (*c*) the inertia forces are proportional to $\frac{w}{g}$, L, d, and $\frac{v^2}{L}$. With these assumptions he has written the equation which states that the ratio of these three pairs of corresponding forces must be the same in the model as in the prototype. The assumption of constant viscosity is also made.

When another pair of resisting forces is introduced, assuming that these forces vary indirectly with the velocity squared, v^2, the area of the stream, $d\,L$, and, inversely, as the hydraulic radius assumed to be proportional to d, another limitation is obtained which states that $L = d$; and then the model law becomes,

$$\frac{1}{d^{1.5}\ s^{0.5}} = \text{a constant} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Thus, for the similarity of all these forces, a scale ratio of $\sqrt[3]{10}$ would be required when the salinity ratio is 10, which is too large to be of much practical value. Fortunately, the authors' experiments have clearly shown that for their problem this additional condition need not be met.

There is one other effect which, under certain conditions, might modify the model law and should receive further study before the law is given a wide application. This effect is mixing. Consider a particle in the fresh water moving over the top of the body of salt water. The impressed force in this case is proportional, not only to the surface gradient (which, in turn, is dependent upon the depth, d), but also upon the density gradient, because both affect the resultant pressure in the direction of motion. The density gradient, of course, is dependent upon mixing. If it can be shown that adherence to the model law will automatically give density gradients either negligible or proportional to the surface gradient, or if, in some other way, it can be shown that all the corresponding important forces are in proper ratio, then the objection just raised would have been overcome. The consistency of the experimental data presented does, in a measure, prove this for the case studied, but any information on the degree of mixing or of salinity that may have been obtained in these experiments, would be a most useful addition to the paper.

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[6] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1047.

As an illustration of the possible effect of mixing upon the model law, let it be assumed that the resistance offered to the motion is dependent, not only upon the area of contact between the two moving fluids, L, the velocity gradient, $\frac{v}{L}$, and the viscosity, but also to some extent upon the differences in density between the two fluids, s. This is reasonable since it is known that the mixing is the greater, the less the difference of the density of the fluids; and, of course, the greater the mixing of the two oppositely moving fluids, the greater the resistance to their motion. The late Kenneth Allen, M. Am. Soc. C. E., has expressed[7] this idea in the statement: "It is clear that salinity of a body of water receiving sewage is an important factor in its diffusion." Let it be assumed that $\frac{1}{s^n}$ should be introduced into Equation (18). Then, it can be shown that the model law would be changed to,

$$\frac{d^{2.5}\ s^{(0.5\,+\,0.5n)}}{L} = \text{a constant}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(19)$$

in which, n is an unknown, but positive, exponent.

In the case studied by Messrs. Rawn and Palmer, the mixing effect was a significant factor. Indeed, its evaluation was the object of their study, and they concluded that no stratification at all is possible if the dilution exceeds 1:200.

Does it not seem plausible that the more rapid decrease in velocity observed for large values of k (when both d and s are small) may have been occasioned by the greater mixing that takes place under those conditions? The effect of a small s upon mixing has just been considered. A small d would give rise to large velocity gradients which, in turn, would seem to promote mixing. Since the forces arising from turbulence have been neglected, in deriving the model law, it would seem that the necessity of providing turbulent conditions in both model and prototype would arise from the requirement of producing similar mixing effects. Surely, if the effect of mixing is important, then immiscible liquids, as mentioned under "Conclusions," could not be used in the model.

The validity of K as a criterion for the case studied would be more convincing if the authors would show the actual computed values for several points in Fig. 7, so that it would be possible to prove from actual data that for radically different values of s and d, with identical values of K, the points as plotted would be substantially coincident.

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[7] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1065.

HERBERT D. VOGEL,[8] ASSOC. M. AM. SOC. C. E. (by letter).—The ideas presented by Professor O'Brien and Mr. Cherno are particularly refreshing at this time in that they serve so admirably to reflect the broader attitude toward design and the use of hydraulic models, that distinguishes the research work of American from European investigators. The writer has frequently urged abandoning the rigid limits of geometric similitude when dealing with models of rivers and other open channels, for the reason that similarity of effect is of infinitely greater importance than a mere similarity of shape. In practice, every model is distorted in some respects, and the investigator must be on guard continually to avoid overlooking the less obvious distortions of properties and forces while attempting to avoid geometric distortion, which in the last analysis can be held responsible for few important errors.

As the authors suggest, the first and most important step in designing the model is to choose a vertical scale that will result in the production of turbulent flow and properly measurable depths. Following this, consideration must be taken of the size of the prototype, the space available for the model, and the water supply at hand. From a practical standpoint these considerations must determine the horizontal scale, which will approximately equal the ratio of the length of available laboratory floor space to the length of the prototype. The distortion will be the ratio between the vertical and horizontal scales, and other scale values must then be determined to conform dynamically with the originally chosen ratios, keeping in mind continually the main purpose of the investigation.

In the case of experiments relative to the motion of salt water through fresh, the O'Brien–Cherno law is provided as a handy tool to determine the salinity ratio necessary for dynamic similarity when horizontal and vertical scales have been established. The law, of course, could be applied conversely to determine the horizontal scale of the model, having already decided upon the vertical scale and salinity ratio, but it is believed that its greatest utility will be in determining a salinity ratio after arbitrarily selecting the linear relationships. An interesting point is raised in conjecturing the practical limits to which the law may be applied, although there appears, offhand, no valid reason for avoiding what might appear to be extreme distortions. In the absence, however, of cogent reasons for resorting to gross geometric distortion and in the interest of conservatism it would seem best to distort no more than is necessary to insure turbulence and measurable depths. Practically every experimenter has laid down limits of distortion that, in his opinion, must not be exceeded, and it is of more than passing interest to note that the imposed limit in each case tends to vary directly with the size of prototype in which the particular experimenter is most interested. Since the writer has to deal largely with flood control and navigation problems relative to the Mississippi River and its tributaries, he is inclined to be rather generous in the matter.

In closing, it is desired to call attention to the notations and symbols used at the United States Waterways Experiment Station, suggesting that their general adoption in the United States might considerably simplify the reading and understanding of technical papers on the subject of similitude. As used by the U. S. Waterways Experiment Station: L_m = a horizontal length in the model; L_n = a corresponding length in Nature (prototype); $l = \frac{L_m}{L_n}$ = the horizontal linear scale; $d = \frac{D_m}{D_n}$ = the vertical linear or depth scale; $v = \frac{V_m}{V_n}$ = the velocity scale; etc. From the foregoing it is seen that capital letters with suitable subscripts represent dimensions in the model and prototype, respectively, while small letters represent their ratios, or the scale values. Continuing: q = discharge scale; a = area scale; e = scale of sand-grain diameters; r = hydraulic radius scale; etc.

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[8] Lieut., Corps of Engrs., U. S. A.; Student, Command and Gen. Staff School, Fort Leavenworth, Kans.

The authors are to be congratulated for the able way in which they have handled a difficult subject and for their courage in daring to deviate from well beaten European paths. The writer has found that the same principles are suitable to investigations of navigation works in rivers[9] and has verified to his own satisfaction the feasibility of designing hydraulic models to preserve dynamic similarity under all conditions of operation.

C. E. GRUNSKY,[10] PAST-PRESIDENT, AM. SOC. C. E. (by letter).—The experiments described in this paper, some of which it was the writer's privilege to have attended, were interesting in giving occular demonstration of the behavior of two vertical surfaces of liquids of different density in contact with each other when freed from restraint. However, the conditions at a lock when a gate between a fresh-water and a salt-water compartment is to be opened, were not reproduced at the model, and it is questionable whether the deductions made by the authors should not be subjected to further review. Personally, the writer does not think it worth while.

In the demonstration with the experimental model the water surfaces on both sides of the partition separating the compartments of different densities were at the same level. There was consequently greater aggregate pressure from top to bottom against the partition on the side of the denser liquid than against the other side. At a lock-gate—the device which is being studied—the opening of the gate does not take place until the total water pressures on the two sides are practically equal. This occurs when the fresh water attains a superelevation over the ocean water of about 1.3 per cent. In a lock with 40 ft of salt water on the sill, the superelevation of the water in the fresh-water compartment would be somewhat more than 0.5 ft.

At the surface of the salt water there would be a velocity head of about 0.52 ft impelling the fresh water against the salt water. At 20 ft above the gate-sill, the velocity head, expressed in fresh water, would be the same on the salt-water side as on the fresh-water side, and at the sill the velocity head, expressed in fresh water, would be 0.52 ft greater on the salt-water side.

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[9] *Civil Engineering*, August, 1932, p. 467.

[10] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif. Mr. Grunsky died on June 9, 1934.

Sudden removal of the partition, of course, would cause an over-run of fresh water in one direction, while there is an under-flow of salt water in the other direction, although necessarily somewhat checked in its velocity of flow by the superelevation of the fresh water, a factor which has not been taken into consideration by the authors.

It is an interesting fact that under certain physical conditions, such superelevation as here referred to may be fairly permanent in locations where there is a gradual transition from fresh water to ocean water. This fact was brought to the attention of the Federal and State engineers who were in charge of studies for a San Francisco Bay salt-water barrier, by the writer in 1931.

Take, for example, any long narrow tidal estuary in which there is a fair depth and considerable tidal movement. Here, as long as there is any material accession of fresh water, there will be a gradual approach, throughout a long stretch of estuary, from fresh-water density to ocean density. If, now, the great block of brackish water in such an estuary is regarded as a block of foreign material interposed between the fresh and the salt water—a gate, in other words, that may be 40 or 50 miles thick up and down stream—then it will at once be apparent that there can be no equilibrium unless the fresh water at the upper end of this block is at a higher level than the salt water at its down-stream end. The brackish water, in other words, has a gradient from its fresher to its saltier end. Despite this gradient, the area of brackish water will be in substantial equilibrium.

Take, for example, the Upper San Francisco Bay region and the delta of the Sacramento and San Joaquin Rivers. At the fall low-water stage of these rivers, brackish water fills Suisun Bay which lies at, or just below, their junction. Brackish water is pushed by the tide back and forth in the San Joaquin River for about 20 or 30 miles in a channel which for a long distance has depths exceeding 50 ft. In this long river stretch, and on down through Suisun Bay and the Straits of Carquinez, there is a gradual increase of water density from fresh water (specific gravity, 1.00) to ocean water (specific gravity, 1.026).

In the Straits of Carquinez below Suisun Bay the water has generally a maximum depth of from 50 to 60 ft. The tidal range here averages, at low stages, 5 to 6 ft, and tidal currents are strong. There is no pocketed salt water. Transition from the fresh to the salt is gradual throughout the stretch of water here under consideration. It follows, therefore, that in the lower delta of these rivers and through Suisun Bay to wherever full-strength ocean water penetrates, there will be an inclined stable water surface with a superelevation at up-stream points of about ($\frac{1}{2} \times 60 \times 0.026 =$) 0.8 ft.

Sea level, for example, approximated by mean tide at Stockton, Calif., above the reach of brackish water, but on the waters of the Lower San Joaquin River, will be about 0.8 ft higher than true sea level because of the depth (60 ft, or thereabouts) throughout which the lighter fresh water must offset the greater pressure of the salt water in the down-stream delta channels.

John B. Drisko,[11] Jun. Am. Soc. C. E. (by letter).—This paper is another striking example of the versatility of hydraulic models. It is also of great value in showing the necessity of a proper choice of model scales if correct results are to be obtained.

The authors introduce their Equation (1) as being equal to "the change of the momentum, $M V_0$, per second", thus ascribing the velocity of the wave front, $V_0 = \frac{dx}{dt}$, to the entire moving mass. This is a bold assumption. If it had been assumed that the velocity decreased slightly from the wave front back toward the undisturbed liquid (that is, in Fig. 6, from B toward Plane C-C), then the momentum of the moving slug $(= \int v\, dM)$ would have been less, yielding a higher value of V_0 in Equation (5). If a higher value of V_0 is correct, then the apparent agreement between the observed results and the (low) values of Equation (5) may be due to the fact that the observed "initial velocity" was measured over a distance long enough to permit of considerable viscous retardation.

Furthermore, if the authors' assumption regarding the velocity is retained, then the kinetic energy of the entire moving mass is,

$$\frac{1}{2} M V^2_0 = \frac{1}{2}\, x\, d\, \frac{w}{g} (G_1 + G_2)\, V^2_0$$

Equating this to the decrease in potential energy (which is $< \frac{1}{4}\, w\, s\, d^2\, x$ since the center of gravity of the salt-water slug, $COBC$, has dropped from a height, $\frac{d}{2}$, to a height greater than $\frac{d}{4}$) gives a result that is quite independent of x, namely, $V_0 < \sqrt{\frac{1}{2}\, \frac{s\, d\, g}{(G_1 + G_2)}}$. This does not agree with observation or with previous calculation and, therefore, the average velocity of the entire moving slug must be less than V_0.

It is interesting to note that Equation (5) is merely the usual equation for the velocity of a wave of translation, $v = \sqrt{gh}$, with the term, $\frac{s}{(G_1 + G_2)}$ included to compensate for the conditions of specific gravity and for the fact that both salt water and fresh water are being accelerated. The wave height, h, is, in this case, equal to $\frac{d}{2}$. Whether or not the absolute value of V_0 calculated by the authors is correct, the form of the expression seems rational, and hence the validity of the model law as derived is in no way affected.

The authors describe the action of a lock full of salt water when released into a larger body of fresh water. The unbalanced force, before release, is the excess of the static pressure on the gate. The distribution of this excess pressure is as shown in Fig. 15. It would be interesting to investigate the reverse condition in which the unbalanced force is in favor of the fresh water, as shown in Fig. 16. As before, the pressure distribution is triangular, but the base of the pressure triangle is at the surface of the salt water instead of at the bottom. The extra height of the fresh water shown could be replaced by a restrained float, in order to reduce to a minimum the quantity of fresh water that would immediately flow over the surface of the salt water.

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[11] Care, U. S. Bureau of Reclamation, Denver, Colo.

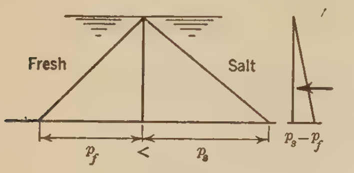

FIG. 15

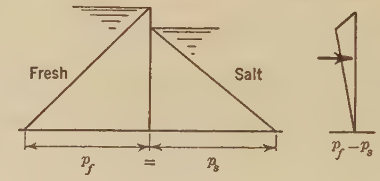

FIG. 16

The statement of the authors, in the last paragraph under "Theory" that "* * * a drop of the surface at any place would have to be made up by a rise of the surface at some other place; * * * and, therefore, no drop of the surface could furnish the energy required to set water in motion", is confusing. If salt water drops and fresh water rises, there is a net loss of potential energy which is available for conversion to kinetic energy.

The derivation of the model law is clear and logical. Of particular significance is the statement following Equation (13) that its physical meaning is that "the inertia, pressure, and friction forces in the prototype must be reduced in the same ratio in the model." Inertia, pressure, and friction are the major forces in this particular problem, and, for true similarity, they must all be given due regard. With some types of models it is difficult or impossible to represent two types of forces in the same model. In the case of ship models, for instance, the skin friction resistance and the wave or form resistance vary according to different laws. In testing ship models, the total resistance is measured, the frictional resistance is calculated empirically, and the difference gives the form resistance of the model. If two types of forces cannot be properly represented in the same model, two models constructed to different scales are sometimes used. In each model, one particular phase of the problem is studied, with little or no regard to the other.

In this paper, the authors have shown how all the essential conditions for a proper simulation of the motion of salt water through fresh may be represented in a single model which will yield correct results.

G. H. KEULEGAN,[12] ESQ. (by letter).—The derivation of model laws in certain cases of fluid motion is effected readily by means of the energy equation. The motion of salt water through fresh presents a clear example. The method, in this particular case, has the advantage of directness and affords a more intimate understanding as to the restrictions to be imposed on the phenomena for a workable model relation than the method of dimensional analysis, for example. This understanding no doubt can also be derived by the method of differential equations; but if that method is used, the rigorous computations are somewhat involved, since it is then necessary to use separately the Eulerian and Lagrangian equations expressed differently for the two liquids, with the additional boundary conditions to be satisfied at the points where these liquids are in contact. The authors use the Eulerian equation in the first stages of the motion (Equations (1) and (9), inclusive). They then revert to the Lagrangian equation, applying it only to the salt water, whereas for a rigid solution of the problem, they should use also a similar equation for the fresh water, together with the boundary conditions existing at the surface of contact. The derivation here presented corrects this defect in the derivation of the authors. It differs from that of the paper only in the definitions of the effective kinematic viscosity and the effective salinity occurring in the expressions of the model law.

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[12] Associate Physicist, Bureau of Standards, U. S. Dept. of Commerce, Washington, D. C.

Certain assumptions are necessary in the present derivation to obtain the law practically in the same form as that given by the authors. It will be assumed first that the motion is essentially a two-dimensional one. In other words, the width of the channel is of the same order as its depth, and the major part of the dissipation of energy arises from the sharp velocity gradient at the boundary separating the two liquids. In this connection it is scarcely necessary to state that the question of surface tension does not enter, since the two liquids mix. On the other hand, the mixing at the surface of separation due to diffusion takes place at a much slower rate than the velocities involved in the flow of the water. Since the motion is assumed to be two-dimensional, unit width of the channel will be assumed in making the energy computations. Other assumptions will be stated as occasion arises.

In addition to the notation of the paper, ρ_1 and ρ_2 denote, respectively, the densities of the salt water and of the fresh water. Suppose that at the time, $t = 0$, the lock holding the salt water is opened and the salt water (shown as the cross-hatched area) has advanced at the time, T, to the position shown in Fig. 17. At the point, P_1, an element of salt water has the mass, $\rho_1\,dX\,dY$, and the elevation, Y_1. Initially, this element occupied the position, P_0, in the lock at the time, $t = 0$, and had the elevation, Y_0. Accordingly, the system suffers a decrease of potential energy,

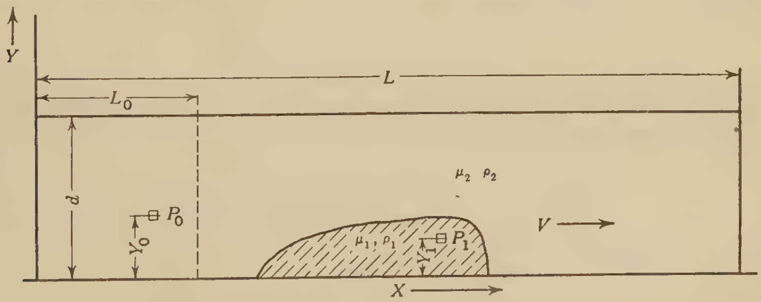

FIG. 17

$$dW = g\,(Y_0 - Y_1)\,(\rho_1 - \rho_2)\,dX\,dY \ldots\ldots\ldots\ldots\ldots (20)$$

arising from this transfer. Thus, at the instant, T, the entire system has suffered a loss of potential energy of amount,

$$W = g\,(\rho_1 - \rho_2) \int_{\text{vol.}} (Y_0 - Y_1)\, dX\, dY \ldots\ldots\ldots\ldots (21)$$

Here the integration is carried out for the region occupied by the salt water. Using the theorem of the mean, Equation (21) becomes,

$$W = g\,(\rho_1 - \rho_2)\,\alpha\, d \int_{\text{vol.}} dX\, dY$$

in which $\alpha\, d$ represents the space average of the drop, averaged over the initial salt-water volume. It follows that,

$$W = \alpha\, g\,(\rho_1 - \rho_2)\, d^2 L_0 \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

since the volume integral is equal to the volume of the lock. It is to be observed that α as derived herein is a pure number having a value which depends on T. Practically, however, α changes very slowly with time. The authors have shown that initially it has the value, 0.25.

This decrease in the potential energy of the system is compensated in part by the increase in the kinetic energy of the system. The remaining part represents energy that is dissipated during the time interval, T. In the computation of the kinetic energy and of the dissipation as well, the effect of the vertical component of the velocity will be assumed to be negligible. With this assumption in mind, the kinetic energy of the system at the time, T, is,

$$K\,E = \tfrac{1}{2} \int_0^d \int_0^{L_1} \rho\, V^2\, dX\, dY$$

in which, V is the velocity at any point along the channel at the time, T. Since ρ does not have the same value throughout, one may write, using the theorem of the mean:

$$K\,E = \tfrac{1}{2}\,\gamma\,\rho_m \int_0^d \int_0^{L_1} V^2\, dX\, dY \ldots\ldots\ldots\ldots\ldots (23)$$

in which, ρ_m represents the value of the densities averaged over the volume of the moving liquids; that is,

$$\rho_m = \frac{\rho_1 + \rho_2}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

and γ represents a number the value of which depends on T. The integrated dissipation function, on the other hand, is,

$$2\,F = \int_0^T \int_0^d \int_0^{L_1} \mu \left(\frac{dV}{dY}\right)^2 dX\, dY\, dt$$

in which, $2F$ represents the energy of dissipation.[13] Since the viscosity does not have the same value throughout, one may write, using again the theorem of the mean:

$$2\,F = \beta\,\mu_m \int_0^T \int_0^d \int_0^{L_1} \left(\frac{dV}{dY}\right)^2 dX\, dY\, dt \ldots\ldots\ldots\ldots (25)$$

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[13] Lamb's "Hydrodynamics," Fourth Edition, p. 549.

in which, μ_m represents the value of the viscosities, averaged over the volume of the moving liquids; that is,

$$\mu_m = \frac{\mu_1 + \mu_2}{2} \ldots\ldots\ldots\ldots (26)$$

μ_1 and μ_2 being the viscosities of the salt water and of the fresh water, respectively, and β representing a number the value of which depends on T. Although rigorously β and γ are dependent on time, their variation with time will be small when the differences, $\mu_1 - \mu_2$ and $\rho_1 - \rho_2$, are small.

The principle of the conservation of energy gives,

$$\alpha\, g\,(\rho_1 - \rho_2)\, d^2 L_0 - \beta\, \mu_m \int_0^T \int_0^d \int_0^{L_1} \left(\frac{dV}{dY}\right)^2 dX\, dY\, dt$$
$$- \tfrac{1}{2}\, \gamma\, \rho_m \int_0^d \int_0^{L_1} V^2\, dX\, dY = 0 \ldots\ldots\ldots\ldots (27)$$

Introducing the initial velocity, V_0, of the front of the salt-water wave and writing:

$$x = \frac{X}{L_0};\ y = \frac{Y}{d};\ \tau = \frac{t}{T};\ L_1 = m L_0;\ v = \frac{V}{V_0};\ \nu = \frac{\mu_1 + \mu_2}{\rho_1 + \rho_2};\ \text{and } s = \frac{\rho_1 - \rho_2}{\rho_1 + \rho_2},$$

the energy equation (Equation (27)) becomes,

$$2\, \alpha\, g\, s\, d - \beta\, \frac{V_0^2\, T\, \nu}{d^2} \int_0^1 \int_0^1 \int_0^m \left(\frac{dv}{dy}\right)^2 dx\, dy\, d\tau$$
$$- \gamma\, \frac{V_0^2}{2} \int_0^1 \int_0^m v^2\, dx\, dy = 0 \ldots\ldots\ldots\ldots (28)$$

The corresponding equation for the model is,

$$2\, \alpha'\, g'\, s'\, d' - \beta'\, \frac{V_0'^2\, T'\, \nu'}{d'^2} \int_0^1 \int_0^1 \int_0^m \left(\frac{dv}{dy}\right)^2 dx\, dy\, d\tau$$
$$- \gamma'\, \frac{V_0'^2}{2} \int_0^1 \int_0^m v^2\, dx\, dy = 0 \ldots\ldots\ldots\ldots (29)$$

If T' of the model is the time corresponding to T of the prototype, the integrals in Equations (28) and (29) are equal in value, and, also, $\alpha = \alpha'$, $\beta = \beta'$, and $\gamma = \gamma'$, approximately, since vertical velocities are neglected. Hence, the following relations are valid:

$$\frac{g\, s\, d}{g'\, s'\, d'} = \frac{V_0^2\, T\, \nu\, d'^2}{V_0'^2\, T'\, \nu'\, d^2} = \frac{V_0^2}{V_0'^2} \ldots\ldots\ldots\ldots (30)$$

If T and T' are corresponding times, it can be shown that $T = \dfrac{L_0}{V_0}$, $T' = \dfrac{L'_0}{V'_0}$, so that instead of Equations (30), the following may be written:

$$\frac{g\, s\, d}{g'\, s'\, d'} = \frac{\nu\, V_0\, L_0\, d'^2}{\nu'\, V'_0\, L'_0\, d^2} = \frac{V_0^2}{V_0'^2} \ldots\ldots\ldots\ldots (31)$$

and, therefore,

$$\frac{\nu\, L_0}{g^{0.5}\, s^{0.5}\, d^{2.5}} = \frac{\nu'\, L'_0}{g'^{0.5}\, s'^{0.5}\, d'^{0.5}} = \kappa \ldots\ldots\ldots\ldots (32)$$

in which, κ is a constant. This agrees in form with the model law obtained by Messrs. O'Brien and Cherno, except that, in Equation (32), ν represents the ratio of $\mu_1 + \mu_2$ to $\rho_1 + \rho_2$, and s the ratio of $\rho_1 - \rho_2$ to $\rho_1 + \rho_2$; whereas, in the law as stated by them, ν is the kinematic viscosity of the salt water and s the difference between the specific gravity of the salt and fresh water.

The foregoing derivation of the model law has been presented primarily because of its theoretical significance. In experiments in which the differences, $\rho_1 - \rho_2$ and $\mu_1 - \mu_2$, are small, as is the case here, and in which the uncertainties in the observation of the position of the salt-water wave are relatively large, it makes little difference whether the model law developed by the authors, or the law developed in this discussion, is used. Although the authors do not discuss their errors of observations, it seems probable that these were large enough to make it impossible to distinguish which model law agrees better with the experimental data.

MORROUGH P. O'BRIEN[14] AND JOHN CHERNO[15], ASSOC. MEMBERS, AM. SOC. C. E. (by letter).—In regard to Lieut. Vogel's suggestion that the symbols used in model work be standardized, the writers agree heartily. The symbols in this paper were changed to conform to those of the Society's Special Committee on Irrigation Hydraulics, in so far as was possible.

The experiments under discussion were made for two purposes, as follows: (1) To study the motion of salt water through fresh water, particularly with reference to the dilution of the salt water during the process; and (2) to discover the model law for this phenomenon which was then to be used to design a model of a part of San Francisco Bay.

This paper is concerned only with the second objective, and whether the water surfaces were equalized on the two sides of the gate does not affect the validity of the resulting model law. Mr. Grunsky is not correct in assuming that lock-gates or lock operation was being studied in these tests. Conditions in actual locks are further complicated by the overfilling of the chamber resulting from the inertia of the water in the culverts; by the non-uniform distribution of salinity in the lock chamber; and by the residual motion of the water in the lock resulting from the filling process. While none of these factors is of major importance in the general characteristics of the motion after the gate is opened, they may disturb the motion in an actual lockage, and the formulas given in the paper must not be used blindly.

The writers cannot agree to Mr. Grunsky's "substantial equilibrium" in the brackish waters of an estuary (disregarding tidal effects) unless the word, "substantial," is changed to "dynamic" or "statistical." The surface gradient would cause water near the surface to flow down stream and the excess pressure due to the salt water would cause an under-flow up stream. The salt water, being diluted with the overlying fresh water, will become less and less brackish during its up-stream movement and, at the same time, the fresh water moving down stream will increase in salinity. As much of the surface gradient as is not used to move the net river flow will serve to maintain these two opposed currents.

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[14] Assoc. Prof., Mech. Eng., Univ. of California, Berkeley, Calif.

[15] Sacramento, Calif.

In a tidal estuary it is difficult to measure these currents, but it is known from current meter measurements that "the flood tide comes in at the bottom;" that is, the ebb current turns to flood current sooner at the bottom than at the surface, the time lag being as much as two hours in some cases. This agrees with the conception of two separate and opposed currents. The surface gradient in a tidal estuary resulting from varying salinity cannot be obtained by simply comparing the elevation of mean sea level at two points because a gradient would exist also with no river inflow, and with constant salinity. This gradient must be great enough to discharge the tidal prism at ebb tide, and this same surface slope tends to decrease the volume of the flood tide. The measured gradient in a tidal river comprises the gradients resulting from net discharge, varying salinity, and tidal flow. Mean sea level, as defined by the U. S. Coast and Geodetic Survey, at Stockton, Calif. (92 miles from the Golden Gate) was 1.5 ft above mean sea level at the Golden Gate in August, 1930; in the same month, mean sea level at Dumbarton Bridge (33 miles from the Golden Gate), was 0.4 ft above the same datum in spite of the fact that there is practically no variation in salinity and no fresh-water inflow in the southern arm of San Francisco Bay.

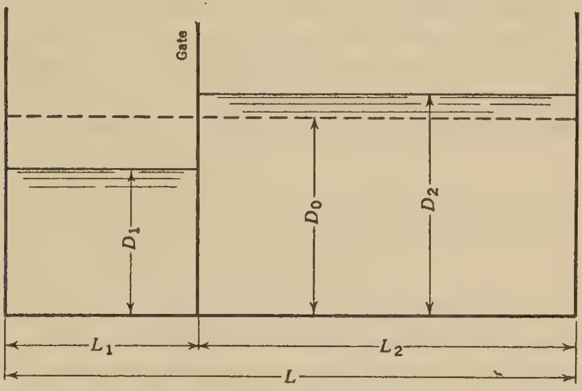

Fig 18

Some experiments were made that have a bearing on Mr. Drisko's question regarding the effect of different water-surface elevations on the two sides of the gate. Referring to Fig. 18, the surfaces will ultimately come to rest at a depth, D_0, given by,

$$D_0 = \frac{1}{L}(D_1 L_1 + D_2 L_2) \quad \dots\dots\dots\dots (33)$$

If the gate is moved to the left (for the conditions shown) until this depth, D_0, is obtained, the length of the lock will be,

$$L_0 = L_1 \frac{D_1}{D_0} \quad \dots\dots\dots\dots (34)$$

If the modulus, K, is computed from these values of L_0 and D_0 and the actual S_0, the velocity curve drawn using Fig. 13 agrees quite well with the experimental values, particularly during the later stages of the motion.

In reply to Mr. Howland's inquiry, the surface slope shown in Fig. 6 existed at the heads of the fresh-water and salt-water waves. Between the two, the interface was practically horizontal provided the lock was long enough (compared with its depth) to make this surface form possible for any length of time. After fresh water reached the closed end of the lock, the salt-water slug had the form shown in Fig. 7. The center of gravity of the salt water moves toward the right, but the edges of the salt slug tend to spread in all directions from the center of gravity. Spreading at the front edge is resisted by the internal friction, but spreading backward reduces the velocity, thus giving the peculiar shape shown. After this shape is reached, the motion does not continue primarily under the action of a pressure gradient, as suggested by Mr. Howland, but rather on account of inertia, until kinetic energy is dissipated.

Motions at C and E, such as described by Mr. Howland, were not observed. Floats placed on the surface to the left of C acquired full velocity toward the left very abruptly, and the same phenomenon was observed by one of the writers in the Ballard Locks, at Seattle, Wash.

The significance of K may best be grasped by writing:

$$K' = \frac{L_0\,\nu}{g^{0.5}\,s^{0.5}\,d^{2.5}} \propto \frac{\nu}{V_0\,d}\left(\frac{L_0\,d\,\dfrac{w}{g}\,s}{\dfrac{1}{2}\,w\,d^2\,s}\right) = \frac{1}{R}\left(\frac{M_s}{F_s}\right) \quad \text{..........(35)}$$

in which, M_s is the mass of salt (not salt water; note that the only net effect of opening the gate is a displacement of the salt toward the right); F_s is the force causing motion; and, R is a Reynolds number. The Reynolds number indicates the relative importance of inertia and friction forces, large values of R corresponding to motions predominated by inertia effects and the reverse. The factor, K', therefore, becomes large when a large mass of salt is to be moved by a small force under conditions conducive to large frictional resistance. The curves corresponding to large values of K in Fig. 7 show a rapid decrease in velocity for precisely this reason. (In tests of this type larger dilution values can be obtained if K is small.)

It should be pointed out here that μ was not defined as the absolute viscosity (see following Equation (10)), but as a coefficient of internal friction and that the conclusions state that the internal friction behaves in a manner similar to the viscous force of laminar flow, but nothing was said as to the value of μ or ν. The theory of turbulent flow as developed by Prandtl, von Karman, and G. I. Taylor expresses the frictional resistance in terms of transfers of momentum or vorticity which results in a "mechanical viscosity" having properties similar to the viscosity of laminar flow. In fact, in the original draft of the paper, ν was called the "mechanical viscosity," but the term was dropped to avoid lengthy explanation of its significance. The important point is that the dilution or mixing is an integral part of the mechanism of turbulent friction and the existence of turbulent mixing was assumed throughout. A model law which represents the motion accurately, particularly the dissipative effect of internal friction, must also represent the mixing. For this reason, the use of mercury in a model is perhaps inadmissible, but the point should be investigated.

Dr. Keulegan's more rigorous derivation of the model law from the standpoint of energy is interesting and it is unfortunate that the accuracy of the experiments does not warrant a comparison of this model law with the simplified law proposed by the writers. Equation (5) was originally derived from the standpoint of energy, and this derivation is perhaps preferable to the momentum method, which is open to criticism from a mechanical standpoint.

Mr. Howland suggests that the motion be considered as the result of the action of a force equal to $\frac{1}{2}\ w\ s\ d^2$ tending to move the entire mass of water between Sections C-C and E-E toward the right, and infers that this tendency must be accompanied by a vertical movement of the free surface. Apparently, he overlooks the fact that the salt water to the left of C has a downward acceleration which reduces the pressure below the hydrostatic pressure corresponding to the depth. Pressure adjustments are propagated with the velocity of sound, which in the present problem is equivalent to occurring instantaneously. As soon as motion of salt water begins, vertical accelerations at C reduce the pressure and the fresh water begins to move toward the left.

In obtaining Equation (2), the force acting was based upon the difference in density, and the momentum was taken as the arithmetic rather than the vector sum of the momenta of the salt and fresh water, even though the two are oppositely directed. This is necessary because the liquid between Sections C-C and E-E is not a free body in the ordinary sense, but rather the salt or fresh water moves subject to the restraint imposed by the presence of the other fluid. The condition is entirely analogous to the oscillations of liquid in a **U**-tube manometer. The activating force results from the unbalance of the columns but the momentum is the arithmetic sum of the momenta in the two legs and not the vector sum. If the vector sum of the momenta was taken in Equation (2), the initial velocity, V_0, would be found to depend only on the depth and the acceleration of gravity and would be independent of the difference in density. Equation (5) can also be obtained by considering the salt (not salt water) as a free body of weight, $w\ s$, and mass, $\frac{w}{g}\ (G_1 + G_2)$ per cu ft. This method is reasonable since the net effect of opening the gate is simply a transfer of salt toward the right.

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WIND STRESS ANALYSIS SIMPLIFIED

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WITH DISCUSSION BY MESSRS. FRANCIS P. WITMER, ELMER K. TIMBY, N. A. RICHARDS, JOHN B. LETHERBURY, FREDERICK MARTIN WEISS, RAYMOND C. REESE, A. FLORIS, ROBINS FLEMING, J. D. GEDO, H. V. SPURR, JOHANNES SKYTTE, JOHN B. WILBUR, AND L. E. GRINTER.

SYNOPSIS

One of the most difficult problems that the structural engineer must solve is the analysis of stresses in a continuous frame in which joint displacements occur. In many cases, since the loading is fixed, the analysis does not involve placing loads for maximum stress. The calculation of wind moments in tall building frames is such a problem.

It is the purpose of this paper to present a method, or rather a system of analysis, by which such frames may be analyzed with satisfactory accuracy in a simple manner and in a fraction of the time required by most exact or nearly exact methods. This simplified method of analysis is made dependent upon the system of balancing fixed-end moments introduced[2] by Hardy Cross, M. Am. Soc. C. E. Accordingly, the application of the method requires the use of only the simplest arithmetic. The importance of this fact is great since the designer's mind is left largely free to study the matter of proportioning the structure.

METHODS IN COMMON USE

An exact analysis of a continuous frame, such as a wind bent, has usually required the solution of a group of simultaneous equations, which is a rather tedious process even when simplified to the greatest possible extent. The

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NOTE.—Presented at the meeting of the Structural Division, New York, N. Y., January 19, 1933.

[1] Prof., Structural Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[2] "Analysis of Continuous Frames by Distributing Fixed-End Moments," *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1.

most usable of these older methods is probably the Slope Deflection Method as advanced by Wilson and Maney.[3] Simplifications of the slope deflection method and of other exact methods have been proposed. Usually, these simplifications have involved the idea of neglecting the effect of moments in one story upon the moments in all other stories except those adjacent to the one under consideration. The approximate methods that resulted were entirely satisfactory except for the time consumed in the analysis. Particularly in the case of wind stresses in tall building frames the time required for an exact or nearly exact analysis has seemed excessive, and engineers have continued to make wide use of the so-called statically determinate methods. This fact is surprising because these methods have been shown to be in error as much as 100%, even for quite regular buildings.

To date, the most usable method of wind stress analysis, which also can claim to be more than a rough estimate, is that suggested by Ross and Morris.[4] This method is both fast and simple to apply, but gives results that seem to be too greatly in error for careful design. In several cases in the two analyses presented by those writers (the Wilson-Maney bent and the American Insurance Union Building), the moments are shown to vary from the correct values by from 15 to 48 per cent.

The most recently suggested method of analyzing continuous frames is that made by Professor Cross.[2] The Cross method of balancing fixed-end moments is directly applicable to all frames when translation occurs at none of the joints. It is fast and simple, and produces any degree of accuracy desired. The writer considers it to be the most satisfactory method yet proposed for the analysis of frames in which the joints do not move. For cases in which joint translations occur, Professor Cross suggests a method of balancing shears, but this, like the other exact methods of caring for joint movements, requires considerable time. Certainly, for the case of wind stress analysis the time required will be found to be excessive.

General Method of Successive Corrections

The writer has suggested a general method of analyzing continuous frames for fixed loads, which he has been in the habit of calling "Successive Corrections."[5] This method was re-stated and examples of its application to wind stress analysis have been presented by Professor Morris.[6] The first step in the general method of successive corrections is to balance fixed-end moments. A simplification is possible in certain cases and this is the "simplified method of wind stress analysis" suggested in this paper. The simplified method is also applicable to certain structures other than building frames, notably bents, open web trusses, and articulated trusses for secondary stress analysis.

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[3] "Wind Stresses in the Steel Frames of Office Buildings," by W. M. Wilson and G. A. Maney, Members, Am. Soc. C. E., *Bulletin No. 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[4] "The Design of Tall Building Frames to Resist Wind," by Albert Ward Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E,, May, 1928, Papers and Discussions, p. 1595.

[5] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 11.

[6] *Loc. cit.*, p. 66.

The general method of successive corrections is re-stated herein to form a background for the presentation of the simplified method, as follows:

(*a*) The structure is allowed to deflect under the loads while the joints are fixed against rotation, and the fixed-end moments are calculated.

(*b*) The fixed-end moments are balanced around the joints until each joint is in equilibrium; that is $\Sigma M = 0$ at each joint. During this process no translations of the joints are allowed. This step is the same as in the Cross method of balancing fixed-end moments.

(*c*) From the balanced moments of Step (*b*) the total shears are determined, and from the difference between these shears and the true shears the artificial restraining forces at the joints are found. (There must be restraining forces since the deflections of the structure as produced in Step (*a*) are not the true deflections.)

(*d*) The joint forces found by Step (*c*) are eliminated by applying equal and opposite joint forces.

(*e*) The structure is allowed to deflect with fixed joints under the action of this new force system, fixed-end moments are calculated, joint moments are balanced, and a second set of joint restraints is computed. It will be found that this second set of joint restraints is smaller and of less importance than the first set.

(*f*) The process is repeated until a final set of joint restraints is determined that is small enough to neglect entirely.

(*g*) The final moments are obtained by adding algebraically the successive sets of balanced moments determined from the original applied loads and the several sets of forces that balance the artificial joint restraints.

The foregoing explanation of the full method of successive corrections is given in detail, in order to make the process clear. When stated in this manner, the reader should visualize clearly a physical action of the structure that corresponds to each step of the process. In practical applications the method may be simplified considerably. Since the first balancing of fixed-end moments is disrupted by the application of a second set of fixed-end moments, it is clear that the first balancing need not be carried to its normal conclusion. In fact, it would be a waste of time to balance the individual joints more than twice before releasing joint restraints. Professor Morris[6] has suggested releasing joint restraints after each balancing of the individual joints, which is entirely satisfactory.

Furthermore, there is an obvious simplification in the practical application of Steps (*c*), (*d*), and (*e*), that is, in releasing joint restraints. It is unnecessary actually to calculate the joint restraints at all, because the difference between the sum of all balanced column moments in a given story and the sum of all the original fixed-end moments is equal to the sum of the second set of fixed-end moments to be applied in this story. This total fixed-end moment is to be divided among the columns in proportion to their *K*-values. Nevertheless, it is of importance that one realize the full significance of this simplification.

The Simplified Method

In the method of successive corrections, the first set of balanced joint moments would have been the true final moments if the first set of artificial joint restraints had been negligible. Moreover, the first set of joint restraints would have been negligible if the deflections of the structure with joints fixed against rotation had been essentially the same as the true deflections. In other words, if the true deflections of the joints can be guessed at, estimated, or actually calculated, it is permissible to force the structure into this deflected position with fixed joints. Then one can calculate the fixed-end moments caused by these deflections and balance the joint moments. If the estimated deflections are reasonably correct, the joint restraints will be found negligible, and the first set of balanced moments may be taken as the final moments. When the first set of joint restraints is too large to neglect, but is still small in comparison with the actual loads, the moments may be corrected approximately. This is accomplished by assuming that there will be no change in the location of the points of contra-flexure in the members. Hence, the final column moments are obtained simply by multiplying the first balanced moments by the ratio of the true shear to the computed shear in that story. Final girder moments are obtained by dividing the sum of the corrected column moments at each joint between the girders in the same ratio as that of the uncorrected girder moments. An outline of this simplified method is, as follows:

(1) Guess at, estimate, or calculate the displacement of each joint of the frame as caused by the applied loads.

(2) Compute the fixed-end moments in the members as caused by these movements. If the translation is horizontal, there are no fixed-end moments in horizontal members.

(3) Balance the moments in succession at all joints until the joints are fully balanced; that is, until $\Sigma M = 0$ at each joint.

(4) Compute the total shear in each story from the balanced moments. The difference between these shears and the real shears caused by the loads is a fair indication of the error in the moments.

(5) Determine the final column moments by proportion, using the relation:

$$\text{Final moment} = \text{Balanced moment} \times \frac{\text{True shear}}{\text{Calculated shear}}$$

(6) Determine the final girder moments by dividing the sum of the corrected column moments at each joint between the girders in the same proportion as that of the uncorrected girder moments.

When the calculated shear is within 10% of the true shear, one need have no fear but that the final moments will check the true moments within a range of error of about 5% when proportioned as indicated in Steps (5) and (6). This factor of 5% is intended to apply to moments of average intensity. A particularly small moment may be in error by 6 or 7 per cent. In several cases the writer has been able to estimate the deflections of building frames with sufficient accuracy so that the calculated shears have been within 6%

of the true shears, and when corrected by proportion the final moments have checked the true moments within 3%, or less. When the frame is irregular, larger errors are naturally to be expected.

The remainder of this paper will be devoted to a development and application of simple methods of estimating the absolute or relative joint deflections of frames before the division of the total moment among the members is known. Relative deflections are as satisfactory as absolute deflections since the final moments are obtained by proportion. Examples of the application of the simplified method to several continuous frames will be given. The examples chosen have previously been analyzed in the literature of this field so that the percentages of error may be studied. The major emphasis is thrown upon the analysis of wind moments in tall building frames because this is the most important of all problems in the field of continuous frames and the one that has been most neglected.

RE-STATEMENT OF THE PROCESS OF BALANCING MOMENTS

In the process of balancing moments, one starts with a set of fixed-end moments in the members. These moments may be caused by intermediate loads or by joint translations. Those caused by intermediate loads are calculated as in any fixed-end beam. When joint translations occur, the fixed-end moments may be computed as follows: In Fig. 1, the deflection at B from a tangent to the elastic curve at A is equal to the statical moment of the $\frac{M}{EI}$ diagram from A to B about B. Therefore,

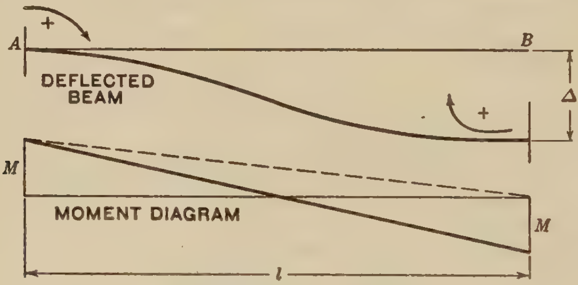

FIG. 1.—JOINT TRANSLATION.

$$\Delta = \frac{Ml}{2EI} \times \frac{2}{3}\, l - \frac{Ml}{2EI} \times \frac{1}{3}\, l = \frac{Ml^2}{6EI} = \frac{Ml}{6\,EK} \quad \ldots\ldots\ldots(1)$$

and the fixed-end moment is,

$$M = \frac{6\,EK\,\Delta}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

The question of signs must be given careful consideration since the sign convention adopted is different from that used by Professor Cross. However, a single statement of this sign convention will cover all cases: When an end moment tends to rotate a joint in a clockwise direction, that moment is always considered positive. In Fig. 1, since the moment in the member, $A\,B$, at A tends to rotate the joint clockwise, this moment is positive; also, the moment at B in the member, $A\,B$, tends to rotate Joint B clockwise, and, therefore, this moment is positive. The deflected shapes of members with the usual conditions of fixed-end moments are shown in Fig. 2. It is possible to find combinations of the cases shown, as, for instance, normal translation plus intermediate loads.

When the fixed-end moments in all members have been determined, the joints are balanced in any desired succession until each joint is eventually freed from any restraining moment. The sum of the moments recorded around any joint at one time is equal to the total unbalanced moment. The individual moments are in proportion to the K-values of the members meeting at the joint. The balancing moments to be recorded at any one time around a joint are of the same sign and of opposite sign to the unbalanced moment.

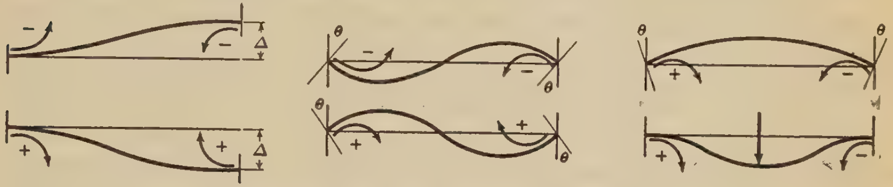

FIG. 2.—SIGNS OF FIXED-END MOMENTS.

Carry-over moments are of the same sign as the balancing moments. They are equal numerically to 50% of the balancing moments if the members are of uniform cross-section from end to end. The algebraic sum of the original fixed-end moment, the balancing moments, and the carry-over moments for any member at a given joint represents the final balanced moment at that end of that member. When this sign convention is used, the algebraic sum of all final balanced moments around a joint is equal to zero; that is, the joint is in static equilibrium.

SECONDARY STRESSES IN TRUSSES

The most obvious application of the simplified method of analysis is in computing the secondary stresses in a truss. In this structure it is not necessary to guess at, or estimate, the deflections because they are caused almost entirely by the direct stresses and may be calculated directly. The computations for deflections may be made either by the Williot-Mohr diagram, or by the algebraic methods of virtual work or elastic weights. The relative deflection of the two ends of each member taken normal to its direction may be obtained from these data, and the fixed-end moments may be calculated by use of Equation (2).

The final secondary moments are obtained by balancing moments around the joints as in any other frame. The final balanced moments are slightly in error because of the existence of small joint restraints. If one is interested in the magnitude of these joint forces, they may be calculated.

As an example of the application of the method, the secondary moments are computed in Fig. 3 for a simple truss analyzed[7] by M. S. Ketchum, M. Am. Soc. C. E. It will be seen that the moments calculated by the simplified method check Dean Ketchum's results closely. The check would naturally have been exact if inaccuracies of slide-rule and graphical computations had not entered into the work. The shears producing the joint restraints are also calculated from the secondary moments given by Dean Ketchum, and are given in Fig. 3 (*c*). The largest joint force is 156 lb., acting downward at *C*. If the members have reasonable areas of cross-section, this force is considerably less than 1% of the load that would cause the assumed direct stresses in the members. Accordingly, the effect of the artificial joint restraints is negligible in secondary stress analysis and is properly omitted in making such computations.

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[7] "Steel Mill Buildings," by M. S. Ketchum, M. Am. Soc. C. E., Fourth Edition, 1921, p. 309.

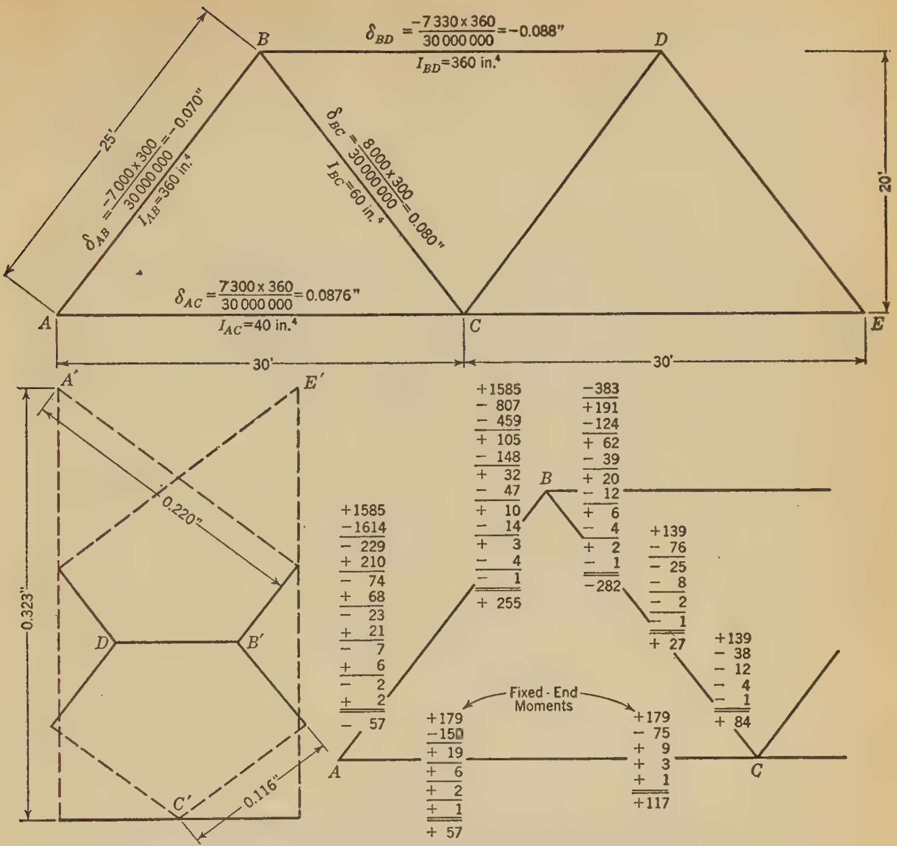
SINGLE-STORY BENTS

All single-story bents may be analyzed by the simplified method without the introduction of any error from neglected joint restraints. Since bents of this kind usually are either unsymmetrical, unsymmetrically loaded, or loaded both vertically and laterally, it becomes necessary to make two separate analyses. Any lateral joint restraint at the top of the bent that appears when balancing fixed-end moments produced by the intra-panel loads (side lurch prevented) is reversed in direction and added algebraically to the real lateral force at the top of the bent. Then, the fixed-end moments produced by this combined lateral force are balanced. A direct proportion exists between these balanced moments and the true moments produced by the lateral force. An explanation may be found in the fact that a single-story bent is limited to one possible type of lateral deflection; that is, side lurch. Hence, the moments are directly proportional to the lateral deflection. The bent will have its proper lateral deflection when the moments are adjusted by proportion so that the bent is in horizontal equilibrum. Final moments are obtained by adding algebraically the moments produced by intra-panel loads and side lurch.

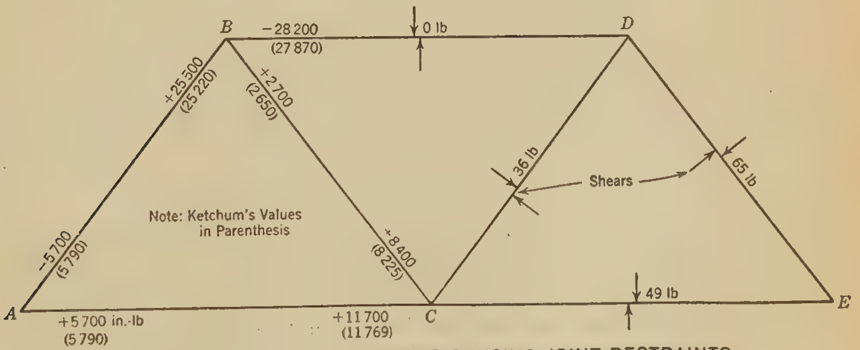
THE MULTIPLE-STORY WIND BENT OF IDEAL PROPORTIONS

As an introduction to the study of wind moments in the tall building frame, the application of the simplified method to the analysis of a tall wind bent of theoretical proportions will be considered. This single-bay structure has two identical columns in each story and any number of stories of constant height. The *K*-values of the columns and girders increase from the top downward in direct proportion to the story shears.

When fixed-end moments caused by the wind shears are balanced, points of contra-flexure will be found at the mid-height of each column and at the center of each girder. The story shears obtained from the balanced moments will be found to bear a constant ratio to the wind shears. Hence, final moments may be obtained by direct proportion. An explanation is found in the fact that the joint restraints, if determined, would form a force system similar in all respects to the original force system. Accordingly, points of contra-flexure would not shift under the application of this second force system. (The local effect of base fixation is not considered here.)


B
D
A
C
E
$\delta_{BD}=\frac{-7\,330\times360}{30\,000\,000}=-0.088''$
$I_{BD}=360$ in.^{4}
25′
$\delta_{AB}=\frac{-7\,000\times300}{30\,000\,000}=-0.070''$
$I_{AB}=360$ in.^{4}
$\delta_{BC}=\frac{8\,000\times300}{30\,000\,000}=0.080''$
$I_{BC}=60$ in.^{4}
$\delta_{AC}=\frac{7\,300\times360}{30\,000\,000}=0.0876''$
$I_{AC}=40$ in.^{4}
20′
30′
30′
A′
E′
D
B′
C′
0.220″
0.116″
0.323″
+1585
−1614
− 229
+ 210
− 74
+ 68
− 23
+ 21
− 7
+ 6
− 2
+ 2
− 57
+1585
− 807
− 459
+ 105
− 148
+ 32
− 47
+ 10
− 14
+ 3
− 4
− 1
+ 255
−383
+191
−124
+ 62
− 39
+ 20
− 12
+ 6
− 4
+ 2
− 1
−282
+139
− 76
− 25
− 8
− 2
− 1
+ 27
+139
− 38
− 12
− 4
− 1
+ 84
Fixed - End Moments
+179
−150
+ 19
+ 6
+ 2
+ 1
+ 57
+179
− 75
+ 9
+ 3
+ 1
+117

(a) WILLIOT DIAGRAM (b) BALANCING MOMENTS


B
D
A
C
E
−28 200
(27 870)
0 lb
+25 500
(25 220)
+2700
(2 650)
−5700
(5790)
+8400
(8225)
36 lb
65 lb
Shears
49 lb
Note: Ketchum's Values in Parenthesis
+5700 in.-lb
(5790)
+11 700
(11 769)

(c) FINAL MOMENTS AND SHEARS CAUSING JOINT RESTRAINTS

FIG. 3.—SECONDARY STRESS ANALYSIS BY SIMPLIFIED METHOD.

It is significant that the side lurch is constant for each story of this bent either when joints are free to rotate or when they are fixed against rotation. The two deflection diagrams may be represented by two straight lines of different slope. A correspondence of deflection diagrams is necessary if exact final moments are to be obtained from the first balanced moments by direct proportion.

WIND STRESSES IN TALL BUILDING FRAMES OF REGULAR PROPORTIONS

A tall building frame will not follow the ideal proportions of a wind bent exactly, but it is of interest that these proportions are followed closely enough so that many buildings approach a straight-line deflection curve under the action of wind loads. With this information as a background, it is possible to set up a system for analyzing wind moments in tall building frames that may be applied to any frame and that is extremely simple when applied to a frame that has a smooth deflection diagram.

First, the assumption is made that the deflections in all stories are in proportion to the story heights. This statement is identical with the mathematical relation, $\frac{\Delta}{h} = C$, in which, Δ and h represent, respectively, the increment of side deflection and the height of any story, and C is a constant. It has been shown that the fixed-end moment is determined from the relation,

$$M = \frac{6\,E\,K\,\Delta}{h} \qquad \text{(3)}$$

Therefore, if $\frac{h}{\Delta}$ is temporarily assumed to be equal to $6\,E$, the fixed-end moments in each column are equal to the K-value of that column.

These fixed-end moments are balanced in the usual manner until all joints are in equilibrium. The resulting story shears are then compared with the wind shears. If the ratios between the calculated shear and the wind shear in all stories are approximately constant—that is, if the variation does not exceed 15%—final column moments may be obtained with satisfactory accuracy by multiplying the calculated column moments in each story by the ratio of the wind shear to the calculated shear in that story. Final girder moments are obtained by dividing the sum of the column moments at a joint between the girders meeting at that joint in the same ratio as that of the uncorrected girder moments.

In certain structures one may find that the ratios of wind shear to calculated shear in adjacent stories correspond with a variation of 10 or 15%, and yet the variation between these ratios for separated stories is much greater. It is well understood that moments applied in one story of a frame have little effect upon moments in any other stories, except those adjacent to the one under consideration. Accordingly, one need compare only the ratios of wind shear to calculated shear in adjacent stories, and if these ratios do not vary from each other by more than 10 or 15%, the moments may be corrected simply by proportion.

If shear ratios in all adjacent stories change in the same direction by 15%, those for the first and sixth stories will vary from each other by 100 per cent. Nevertheless, approximately correct final moments can be obtained by proportion. The reader can prove this statement for himself in a simple manner. Consider a group of stories cut from a frame in which all columns and girders have the same *K*-values, all stories have equal heights, and the wind shear changes from the top downward by increments of 20% of the shear in the top story. If fixed-end moments equal to the *K*-values of the columns are balanced, it will also be found that successive ratios of wind shear to calculated shear increase uniformly from the top downward by increments of 20 per cent. However, when corrected by proportion, the moments are within 11% of the true moments. It follows that a reasonable correspondence (less than 15% variation) between the ratios of actual shear to calculated shear in adjacent stories of a building frame is the only important criterion for determining when final moments may be obtained by proportion.

The foregoing discussion makes it clear that the method of balancing fixed-end moments equal to the *K*-values of the columns is directly applicable to practically any frame that has a reasonably smooth deflection curve even if this curve does not approach a straight line. An interesting example is that of a set-back building. If the parts of the building above and below the set-back are reasonably regular, it is only necessary that one estimate carefully the relative deflections of the first two stories above and the first two stories below this point, in order to set the relative slopes of the two assumed straight deflection lines. (A method of estimating deflections will be developed in a later section of this paper.) From this point the analysis is unchanged except that fixed-end moments in the upper part of the building are equal to the *K*-values of the columns multiplied by the ratio of the slopes of the top and bottom parts of the building. The fixed-end moments in the lower section of the building are taken as equal to the *K*-values of the columns.

TABLE 1.—ESSENTIAL DIMENSIONS AND DATA FOR SOLUTION OF PROBLEMS IN FIGS. 4, 5, AND 8

Story	THE WILSON-MANEY BENT					AMERICAN INSURANCE UNION BUILDING						
	VALUES OF *K*					VALUES OF *K*						
				Ceiling Girders				Inside Columns		Ceiling Girders		
	Story height, in feet	Outside column, *A*	Inside column, *B*	Outside bay, *A-B* (22 ft.)	Center bay, *B-B′* (18 ft.)	Story height, in feet	Outside column, *A*	*B*	*C*	Outside bay, *A-B* (14.33 ft.)	Intermediate bay, *B-C* (11.12 ft.)	Center bay, *C-C′* (21.33 ft.)
1....	22	25.8	25.8	30.5	37.3	22.15	42.98	39.65	59.87	135.85	191.88	293.81
2....	16	35.6	35.6	21.4	29.2	18.58	51.52	63.74	107.93	53.60	69.06	......
3....	14	35.4	35.5	21.4	26.2	12.96	73.88	91.40	154.78	53.60	69.06	......
4....	14	35.4	35.5	19.5	26.2	15.25	62.78	77.67	131.52	135.85	191.88	160.71
5....	14	29.4	30.4	19.5	23.8	11.82	37.84	42.91	59.30	28.81	37.12	37.14
6....	14	29.4	30.4	14.1	17.2	10.58	42.24	47.89	66.19	28.81	37.12	37.14
7....	12	28.7	30.0	12.8	15.7	11.00	40.64	46.07	63.68	28.81	37.12	37.14
8....	12	28.7	30.0	7.7	9.4	10.33	35.01	40.41	55.11	28.81	37.12	37.14

An example of the application of the simplified method in the form suggested herein (balancing fixed-end moments equal to the K-values of the columns) is given in Fig. 4. The frame selected is the lower stories of the Wilson-Maney bent that was analyzed by the slope deflection method in *Bulletin No. 80* of the University of Illinois Engineering Experiment Station. The dimensions of this frame and the K-values of the members are given in Table 1. This structure, being of regular proportions, is well suited to analysis by the simplified method in the form described previously.

A brief explanation of the computations in Fig. 4 seems desirable. The figures enclosed in small boxes at the joints are percentage factors for balancing moments around the joints. The first figure at the top of each row of column moments represents the original fixed-end moment, which is equal to the K-value of the column. There are no fixed-end moments in the girders. The center girders of this symmetrical building are represented as having pins at their mid-points, which are fixed against translation. This represents their true condition and is permissible, provided their K-values are increased 50% when the distribution factors are determined. If this simplification is used, there are no carry-over moments for the center girders. As usual, a single horizontal line indicates that a balancing moment has been recorded immediately above, and two horizontal lines indicate that an entire balancing process has been completed. The final moments, obtained by proportion, are shown in parentheses.

When the ratios of calculated shear to wind shear in each story of the Wilson-Maney bent (Fig. 4), were determined, they were found to correspond within a variation of 11% for adjacent stories. Accordingly, the balanced moments were corrected by proportion, and the final moments will be found to check the true moments as determined by the slope deflection method within a maximum error of 7 per cent. This frame has also been analyzed by the Ross-Morris method and the Fleming Method No. 1, and the reported moments[4] showed maximum errors, respectively, of 15 and 39 per cent. A comparison of the moments obtained from Fig. 4 extended to seven stories, with the exact moments by the slope deflection method, is given in Part (*a*) of Table 2.

INTRODUCTION OF CORRECTION MOMENTS FOR BALANCING SHEARS IN AN IRREGULAR FRAME

If the method of estimating fixed-end moments to be in proportion or equal to the K-values of the columns is used on an irregular frame, one will not find the ratios of wind shear to calculated shear to be alike, even approximately, in all adjacent stories. Since these criterion ratios must be made to correspond, it is necessary to introduce additional fixed-end moments into certain stories to bring these ratios reasonably close together. After estimating the total fixed-end moments required to accomplish this purpose, the fixed-end moment for a given story is divided among the columns in proportion to their K-values. These moments are then re-balanced and the results are added algebraically to the first balanced set of moments. Then, if the

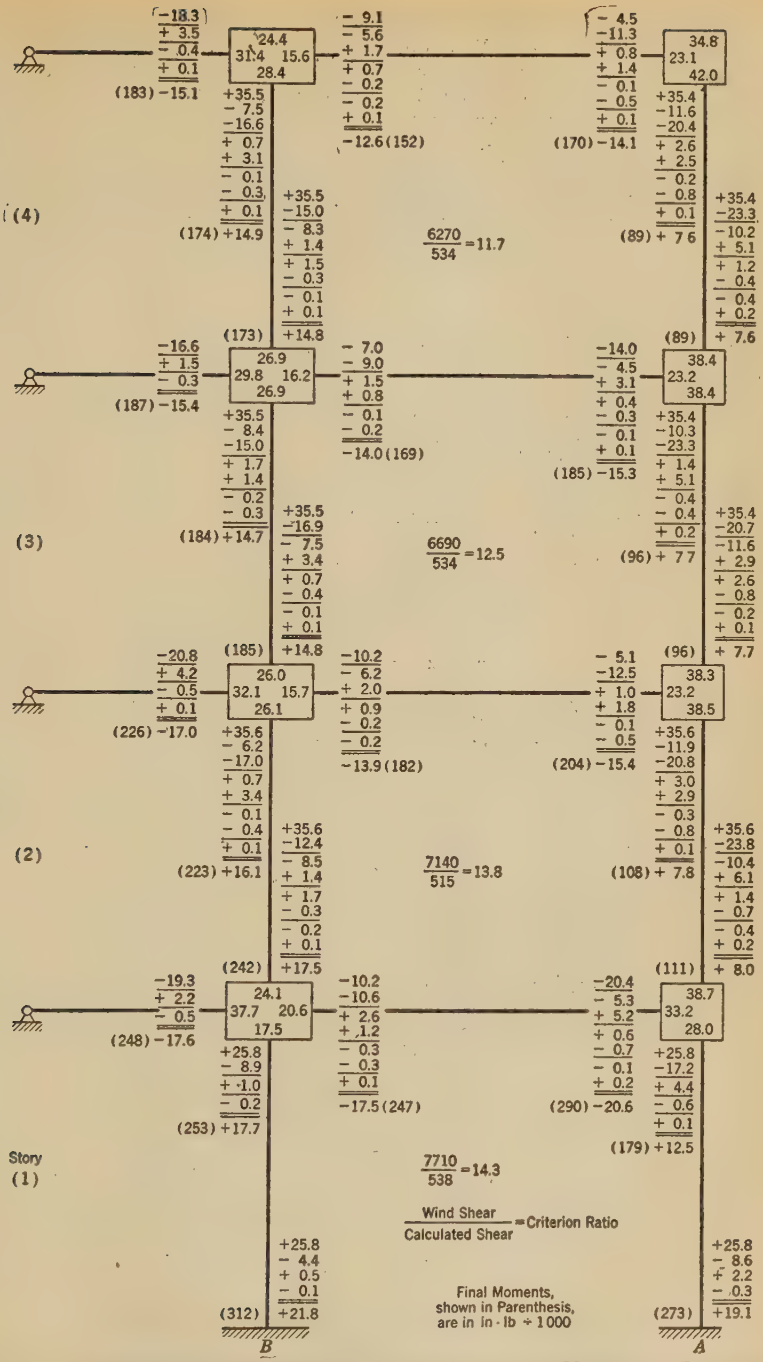

−18.3 +3.5 −0.4 +0.1 (183) −15.1
24.4 31.4 15.6 28.4
+35.5 −7.5 −16.6 +0.7 +3.1 −0.1 −0.3 +0.1 (174) +14.9
−9.1 −5.6 +1.7 +0.7 −0.2 −0.2 +0.1 −12.6 (152)
−4.5 −11.3 +0.8 +1.4 −0.1 −0.5 +0.1 (170) −14.1
34.8 23.1 42.0
+35.4 −11.6 −20.4 +2.6 +2.5 −0.2 −0.8 +0.1 (89) +7.6
(4)
+35.5 −15.0 −8.3 +1.4 +1.5 −0.3 −0.1 +0.1 (173) +14.8
6270/534 = 11.7
+35.4 −23.3 −10.2 +5.1 +1.2 −0.4 −0.4 +0.2 (89) +7.6
−16.6 +1.5 −0.3 (187) −15.4
26.9 29.8 16.2 26.9
−7.0 −9.0 +1.5 +0.8 −0.1 −0.2 −14.0 (169)
−14.0 −4.5 +3.1 +0.4 −0.3 −0.1 +0.1 (185) −15.3
38.4 23.2 38.4
+35.5 −8.4 −15.0 +1.7 +1.4 −0.2 −0.3 (184) +14.7
+35.4 −10.3 −23.3 +1.4 +5.1 −0.4 −0.4 +0.2 (96) +7.7
(3)
+35.5 −16.9 −7.5 +3.4 +0.7 −0.4 −0.1 +0.1 (185) +14.8
6690/534 = 12.5
+35.4 −20.7 −11.6 +2.9 +2.6 −0.8 −0.2 +0.1 (96) +7.7
−20.8 +4.2 −0.5 +0.1 (226) −17.0
26.0 32.1 15.7 26.1
−10.2 −6.2 +2.0 +0.9 −0.2 −0.2 −13.9 (182)
−5.1 −12.5 +1.0 +1.8 −0.1 −0.5 (204) −15.4
38.3 23.2 38.5
+35.6 −6.2 −17.0 +0.7 +3.4 −0.1 −0.4 +0.1 (223) +16.1
+35.6 −11.9 −20.8 +3.0 +2.9 −0.3 −0.8 +0.1 (108) +7.8
(2)
+35.6 −12.4 −8.5 +1.4 +1.7 −0.3 −0.2 +0.1 (242) +17.5
7140/515 = 13.8
+35.6 −23.8 −10.4 +6.1 +1.4 −0.7 −0.4 +0.2 (111) +8.0
−19.3 +2.2 −0.5 (248) −17.6
24.1 37.7 20.6 17.5
−10.2 −10.6 +2.6 +1.2 −0.3 −0.3 +0.1 −17.5 (247)
−20.4 −5.3 +5.2 +0.6 −0.7 −0.1 +0.2 (290) −20.6
38.7 33.2 28.0
+25.8 −8.9 +1.0 −0.2 (253) +17.7
+25.8 −17.2 +4.4 −0.6 +0.1 (179) +12.5
Story (1)
7710/538 = 14.3
Wind Shear / Calculated Shear = Criterion Ratio
+25.8 −4.4 +0.5 −0.1 (312) +21.8
Final Moments, shown in Parenthesis, are in in-lb ÷ 1000
+25.8 −8.6 +2.2 −0.3 (273) +19.1
B
A

FIG. 4.—WILSON-MANEY BENT ANALYZED BY THE SIMPLIFIED METHOD BY BALANCING MOMENTS EQUAL TO THE *K*-VALUES OF THE COLUMNS.

TABLE 2.—Comparisons of Moments as Computed by the Exact Slope Deflection Method and by the Simplified Method

(*a*) Wilson-Maney Bent* (Regular Proportions, See Fig. 4)

Story	Column *A*			Column *B*			Column *C*			Floor	Girder M_{AB}			Girder M_{BA}			Girder $M_{BB'}$		
	Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error		Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error	Slope deflection	Simplified method	Percentage, error
7																			
	60.0	56	7	124.0	123	1	....	....	....	6	135.0	131	3	125.6	123	2	141.5	141	0
	75.0	75	0	143.3	141	2	....	....	....										
6																			
	83.2	84	1	155.7	156	0	....	....	....	5	167.7	170	1	152.0	154	1	166.0	166	0
	84.5	86	2	162.3	164	1	....	....	....										
5																			
	82.8	81	2	162.0	161	0	....	....	....	4	170.5	170	0	153.6	152	1	184.4	183	1
	87.8	89	1	176.1	174	1	....	....	....										
4																			
	87.8	89	1	173.5	173	0	....	....	....	3	187.1	185	1	171.7	169	2	189.6	187	1
	100.3	96	4	187.5	184	2	....	....	....										
3																			
	90.7	96	6	182.0	185	2	....	....	....	2	203.0	204	0	184.0	182	1	226.0	226	0
	113.0	108	4	226.5	223	2	....	....	....										
2																			
	107.5	111	3	239.0	242	1	....	....	....	1	287.8	290	1	245.0	247	1	247.5	248	0
	178.8	179	0	251.0	253	1	....	....	....										
1																			
	272.0	273	0	308.2	312	1	....	....	....	....	....	....	....	....	....	....	....	....	....

(*b*) American Insurance Union Building† (With Correction Moments for Balancing Shears, See Fig. 5)

											Girder *AB*			Girder *BC*			Girder *CC′*		
	183	18	2	382	39	2	439	46	5		365	38	4	400	41	3			
5										6							432	47	9
	196	18	8	389	38	2	452	43	5		332	34	2	416	43	3			
	184	18	2	378	38	1	437	44	1		380	36	5	421	42	0			
4										5							452	44	3
	186	17	9	383	38	1	445	44	1		347	34	2	436	43	1			
	188	20	6	360	37	3	421	43	2		374	37	1	406	41	1			
3										4							441	44	0
	294	29	1	471	46	2	571	55	4		338	34	1	424	43	1			
	362	36	1	678	67	1	825	81	2		655	65	1	631	62	2			
2										3							677	66	3
	239	25	5	526	53	1	427	43	1		518	51	2	719	70	3			
	333	31	7	702	68	3	391	37	5		572	56	2	707	71	0			
M										2							....	....	....
	301	30	0	681	69	1	269	30	12		520	50	4	817	80	2			
	332	34	2	678	66	3	670	64	4		632	64	1	792	79	0			
1										*M*							....	....	....
	400	40	0	802	80	0	1 078	109	1		567	56	1	938	94	0			
	583	57	2	682	67	2	1 000	98	2		983	97	1	745	74	1			
0										1							1 285	128	0
	713	71	0	730	72	1	1 086	108	1		738	73	1	791	79	0			

(*c*) American Insurance Union Building† (Deflections Estimated by Use of n-Values, See Table 3 and Fig. 8)

											Girder *A*			Girder *B*			Girder *C*		
	183	20	9	382	39	2	439	44	0		365	36	1	400	38	5			
5										6							432	42	3
	196	20	2	389	36	7	452	43	5		332	32	4	416	41	1			
	184	19	3	378	37	2	437	44	1		380	39	3	421	40	5			
4										5							452	45	0
	186	20	8	383	37	3	445	44	1		347	33	5	436	42	4			
	188	19	1	360	36	0	421	42	0		374	39	4	406	40	1			
3										4							441	44	0
	294	29	1	471	46	2	571	58	2		338	34	1	424	42	1			
	362	36	1	678	66	3	825	82	1		655	65	1	631	62	2			
2										3							677	68	0
	239	24	0	526	52	1	427	44	3		518	50	3	719	72	0			
	333	33	1	702	71	1	391	38	3		572	57	0	707	71	0			
M										2							....	....	....
	301	30	0	681	69	1	269	26	3		520	52	0	817	82	0			
	332	32	4	678	68	0	670	68	1		632	63	0	792	81	2			
1										*M*							....	....	....
	400	40	0	802	79	1	1 078	107	1		567	56	1	938	94	0			
	583	56	4	682	68	0	1 000	99	1		983	96	2	745	74	1			
0										1							1 285	126	
	713	70	2	730	73	0	1 086	108	1		738	73	1	791	80	1			

* Slope deflection moments were computed by Wilson and Maney and are given in inch-pounds ÷ 1 000.

† Slope deflection moments were computed by Ross and Morris and are given in inch-pounds ÷ 10 000.

criterion ratios for adjacent stories correspond within the desired limits, final moments are obtained by proportion.

Some difficulty may be found in estimating correctly the total fixed-end moments to apply in neighboring stories in order to make the criterion ratios correspond. In selecting the value of the moment to be applied in a given story, one must consider the fact that the final balanced moment will be changed by considerably less than the value of the applied fixed-end moment. It is also of importance that a total positive moment applied in a given story will produce a total negative moment in each adjacent story. An estimate of these effects can be made wherever the distribution factors at the joints are known.

An example of the application of this method to an irregular frame is given in Fig. 5. The frame selected is the lower stories of the American Insurance Union Building. Dimensions and K-values of the members for this building are given in Table 1. This structure was chosen since it has been analyzed previously by three methods, one of which was the exact slope deflection method by Ross and Morris.[4] The fixed-end moments selected for the columns in Fig. 5 are twice the Column K-values. These moments were used in order to increase the accuracy of the final moments without making necessary the use of three-place balancing moments. Since final moments are obtained by proportion, it is permissible to multiply all fixed-end moments by any factor desired.

In order to isolate the lower eight stories, assume the moments coming to these joints to be the same from above and below. Do not transfer the carry-over moment across the girder. These approximations make the moments calculated for the eighth story valueless. The final moments in Fig. 5—in inch-pounds, divided by 100 000—are shown in parenthesis. The reader will note the correction moments that are introduced into Story 2.

The first set of balanced moments for the American Insurance Union Building, (Fig. 5) was used to calculate the story shears, and from these shears the ratios of wind shear to calculated shear in each story were determined. These criterion ratios seemed satisfactory except for the low ratio existing in the first story above the mezzanine, which is called the second story in this building. Accordingly, a total fixed-end moment of —160 was divided among the columns of this story in proportion to their K-values and re-balanced. These fixed-end correction moments are the second figures below the first pair of horizontal lines in each column of moments. The value of —160 was chosen after careful inspection as being the smallest total moment that could be applied in this story and still increase the criterion ratio for this story up to approximately the values found in the adjacent stories. The final results justified the selection of this fixed-end moment although either a slightly larger or a slightly smaller moment would have proved to be nearly as satisfactory.

The final criterion ratios for adjacent stories vary from each other by a maximum of 21% and yet the largest error found in a final moment is 12%, which occurs in one of the smaller moments. It is of interest in this connection that the maximum error found by Ross and Morris was 48% by their

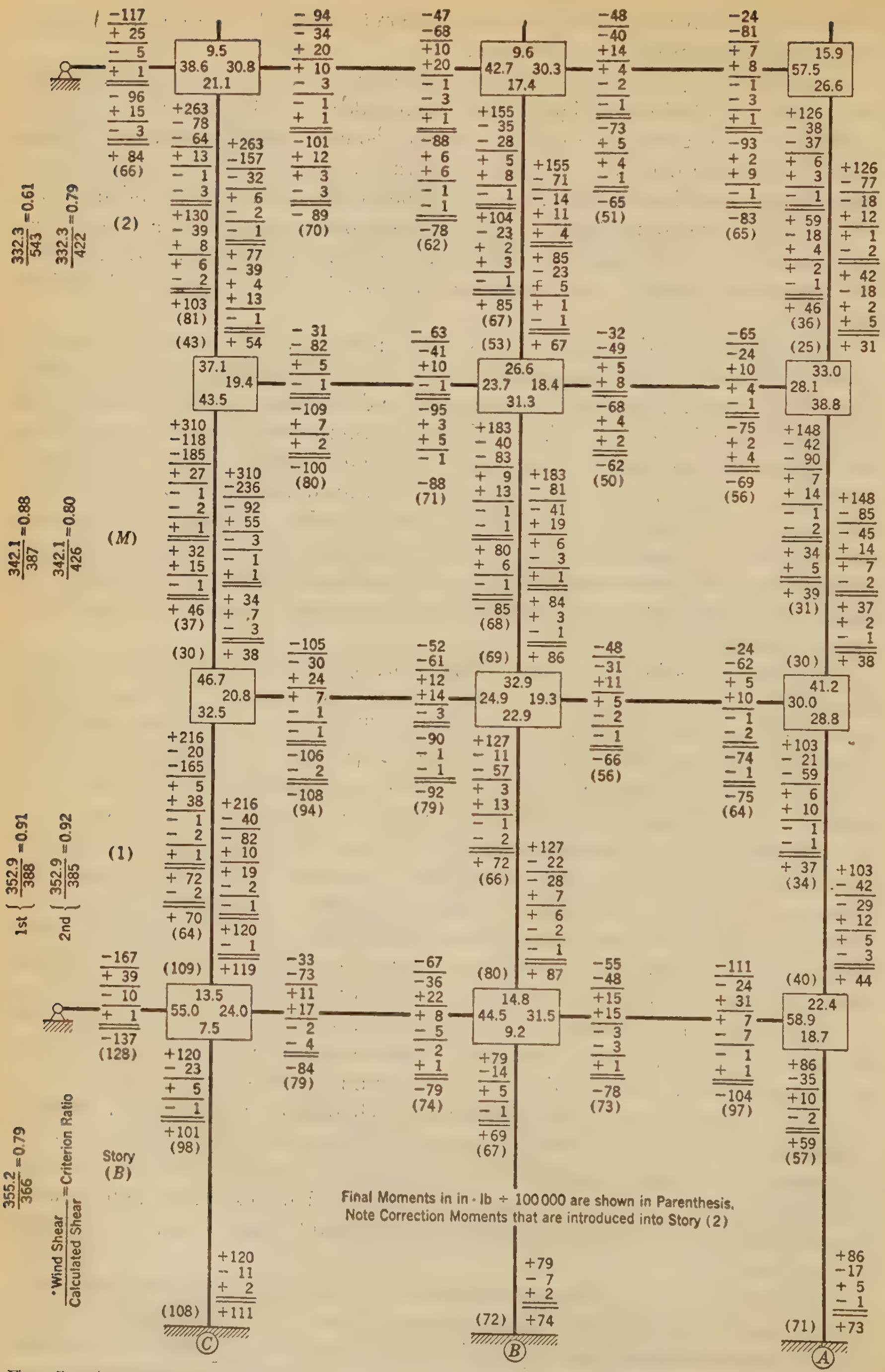
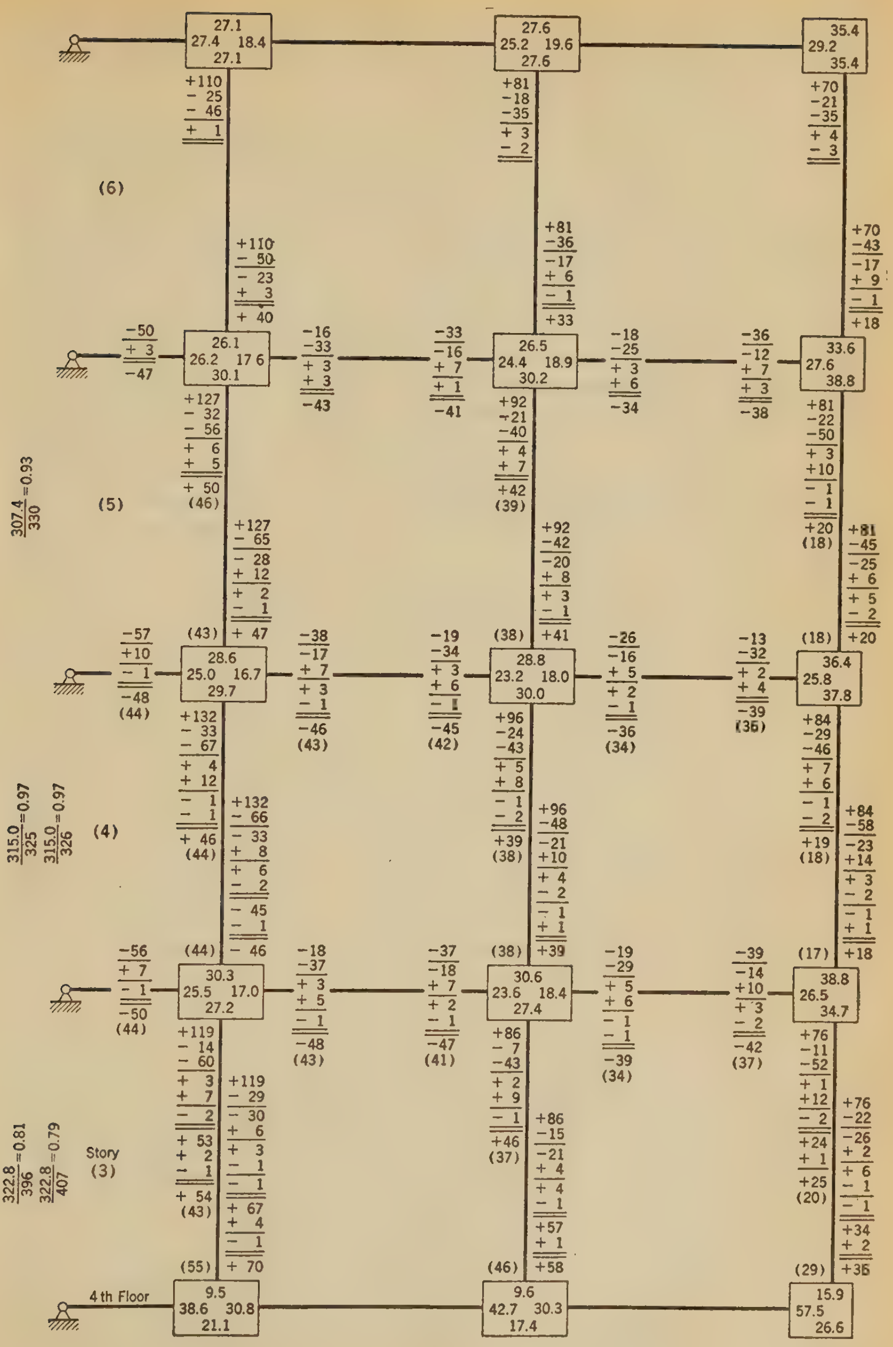

332.3/543 = 0.61
332.3/422 = 0.79
(2)
342.1/387 = 0.88
342.1/426 = 0.80
(M)
1st {352.9/388 = 0.91
2nd {352.9/385 = 0.92
(1)
355.2/366 = 0.79
Story (B)
*Wind Shear / Calculated Shear = Criterion Ratio
Final Moments in in · lb ÷ 100 000 are shown in Parenthesis.
Note Correction Moments that are introduced into Story (2)
C
B
A

FIG. 5.—AMERICAN INSURANCE UNION BUILDING ANALYZED BY THE SIMPLIFIED METHOD CORRECTION MOMENTS APPLIED


27.1 27.4 18.4 27.1
27.6 25.2 19.6 27.6
35.4 29.2 35.4
+110 − 25 − 46 + 1
+81 −18 −35 + 3 − 2
+70 −21 −35 + 4 − 3
(6)
+110 − 50 − 23 + 3 + 40
+81 −36 −17 + 6 − 1 +33
+70 −43 −17 + 9 − 1 +18
−50 + 3 −47
26.1 26.2 17.6 30.1
−16 −33 + 3 + 3 −43
−33 −16 + 7 + 1 −41
26.5 24.4 18.9 30.2
−18 −25 + 3 + 6 −34
−36 −12 + 7 + 3 −38
33.6 27.6 38.8
+127 − 32 − 56 + 6 + 5 + 50 (46)
+92 −21 −40 + 4 + 7 +42 (39)
+81 −22 −50 + 3 +10 − 1 − 1 +20 (18)
$\frac{307.4}{330}=0.93$
(5)
+127 − 65 − 28 + 12 + 2 − 1 + 47
(43)
+92 −42 −20 + 8 + 3 − 1 +41
(38)
+81 −45 −25 + 6 + 5 − 2 +20
(18)
−57 +10 − 1 −48 (44)
28.6 25.0 16.7 29.7
−38 −17 + 7 + 3 − 1 −46 (43)
−19 −34 + 3 + 6 − 1 −45 (42)
28.8 23.2 18.0 30.0
−26 −16 + 5 + 2 − 1 −36 (34)
−13 −32 + 2 + 4 −39 (35)
36.4 25.8 37.8
+132 − 33 − 67 + 4 + 12 − 1 − 1 + 46 (44)
+96 −24 −43 + 5 + 8 − 1 − 2 +39 (38)
+84 −29 −46 + 7 + 6 − 1 − 2 +19 (18)
$\frac{315.0}{325}=0.97$
$\frac{315.0}{326}=0.97$
(4)
+132 − 66 − 33 + 8 + 6 − 2 − 45 − 1 − 46
(44)
+96 −48 −21 +10 + 4 − 2 − 1 + 1 +39
(38)
+84 −58 −23 +14 + 3 − 2 − 1 + 1 +18
(17)
−56 + 7 − 1 −50 (44)
30.3 25.5 17.0 27.2
−18 −37 + 3 + 5 − 1 −48 (43)
−37 −18 + 7 + 2 − 1 −47 (41)
30.6 23.6 18.4 27.4
−19 −29 + 5 + 6 − 1 − 1 −39 (34)
−39 −14 +10 + 3 − 2 −42 (37)
38.8 26.5 34.7
+119 − 14 − 60 + 3 + 7 − 2 + 53 + 2 − 1 + 54 (43)
+86 − 7 −43 + 2 + 9 − 1 +46 (37)
+76 −11 −52 + 1 +12 − 2 +24 + 1 +25 (20)
$\frac{322.8}{396}=0.81$
$\frac{322.8}{407}=0.79$
Story (3)
+119 − 29 − 30 + 6 + 3 − 1 − 1 + 67 + 4 − 1 + 70
(55)
+86 −15 −21 + 4 + 4 − 1 +57 + 1 +58
(46)
+76 −22 −26 + 2 + 6 − 1 − 1 +34 + 2 +36
(29)
4th Floor
9.5 38.6 30.8 21.1
9.6 42.7 30.3 17.4
15.9 57.5 26.6

of Balancing Moments Equal to Twice the *K*-Values of the Columns in Story 2.

own method and 100% by the Fleming Method No. 1. A comparison of the moments obtained from Fig. 5 with the exact moments by the slope-deflection method is given in Part (*b*) of Table 2.

It is important that the reader realize the practical limitations that should be placed upon this method of analysis, in order that he may understand the field of use for the second method which will be described later. The desire of the writer has been to reduce wind stress analysis to the simple process of balancing one set of fixed-end moments and to eliminate the need for balancing shears with the corresponding introduction of a second, third, and fourth set of fixed-end moments which must be balanced in their turn. The foregoing method of analysis is ideal, wherever the frame deflects into a reasonably smooth curve. For such a structure, the analysis is completed by balancing a single set of fixed-end moments and then by correcting the balanced moments by proportion. Even if there is a considerable irregularity in one or two stories, the method is still advantageous since proper correction moments can be introduced into these stories, re-balanced with small effort, and the moments corrected by proportion as for a regular frame. However, if more than two neighboring stories of the frame are highly irregular, it becomes rather difficult to estimate the proper fixed-end correction moments. In fact, one may err to such an extent in selecting these moments that, after re-balancing, the shear ratios for adjacent stories still do not correspond within the desired range of variation, and it becomes necessary to introduce a second set of correction moments. Naturally, such mistakes are most likely to occur in an analysis performed by one who is unfamiliar with the method.

After making use of this method of balancing fixed-end moments in proportion to the K-values of the columns, the reader will find that its field of major importance is in the analysis of those frames for which an inspection of the arrangement of the members and of their K-values indicates that correction moments probably will be required in one or two stories only. If this inspection indicates that correction moments will be required in a large number of stories, it seems desirable to make a closer estimate of the true relative deflections, in order that the introduction of correction moments may be avoided.

WIND STRESSES IN TALL BUILDING FRAMES OF IRREGULAR PROPORTIONS

In the method of analysis just presented, the story deflections of the frame are assumed to be in proportion to the story heights. The analysis of an irregular frame by this system is complicated subsequently by the need for introducing correction moments into one or more stories of the frame. A method of estimating the true story deflections would seem to offer an advantage since the need for the introduction of correction moments might be eliminated in this manner. In fact, final moments can be obtained within an error of about 6% if the side deflections of each story can be estimated within a variation of 10 per cent. The following method of computing deflections has been found to give approximately this accuracy in all except highly irregular frames. As a maximum error of even 10 or 12% in the wind moments would not be objectionable ordinarily, this method may be used in practical design even for extremely irregular structures. Clearly, this system of estimating deflections may be combined with the conception of producing equal criterion ratios for all stories, by the introduction of correction moments into certain stories, to give a method for determining wind moments within any desired degree of precision.

Since the lateral deflections of a building under wind load are dependent upon the properties of all the columns and all the girders, it seems reasonable that the deflections might be estimated with considerable accuracy by considering the deflections of a single combined column restrained by single girders at each floor level. The K-value of the combined column for each story is taken as the summation of the K-values of all columns in that story, or ΣK_c. The combined girder at each floor level has a K-value equal to twice the sum of the K-values of all girders at that floor, or $2 \Sigma K_g$. The factor, 2, is required because each girder is effective in resisting the moments at two joints. The problem is now reduced to estimating the lateral deflections of this single column with single restraining girders at each floor level.

It is necessary, first, to consider the relation between moment, deflection, and joint rotation in a single story. The lateral deflection of the combined column may be looked upon as made up of two parts: (1) The angle change at the far end times the story height; and (2) the deflection at the near end from a tangent drawn at the far end. However, considering the deflection as divided at the mid-height, as shown in Fig. 6, it becomes clear that,

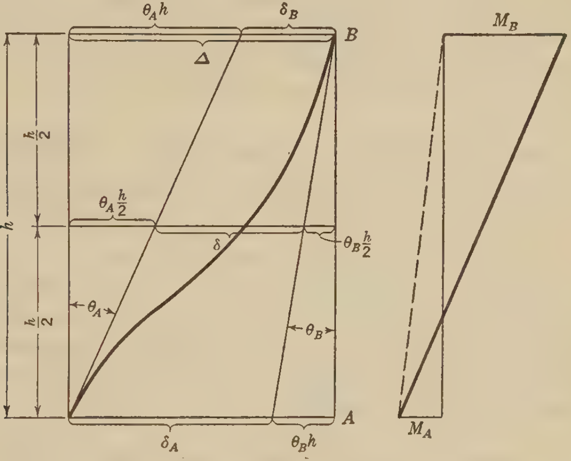


FIG. 6.—COLUMN DEFLECTION.

$$\Delta = \frac{\theta_A + \theta_B}{2} h + \frac{\delta_A + \delta_B}{2} = \theta_{\text{avg}} h + \delta$$

but δ is the average of the deflections at the two ends measured from tangents drawn at the far ends. Hence, δ is readily computed by area moments; thus,

$$\delta = \frac{1}{2EI}\left(\frac{M_A h^2}{6} + \frac{M_B h^2}{6}\right) = \frac{(M_A + M_B) h}{12 EK}$$

The sum of M_A and M_B for the combined column is equal to the total moment, M_t, in the story, which is equal to the shear times the story height, and the K-value of the combined column is ΣK_c. Hence, $\delta = \dfrac{M_t h}{12 E \Sigma K_c}$, and,

$$\Delta = \theta_{avg}\, h + \frac{M_t h}{12 E \Sigma K_c} \quad \dots\dots\dots\dots\dots\dots (4)$$

Equation (4) can also be derived from the slope-deflection equations.

The second term of the expression for Δ is unaffected by the division of M_t into M_A and M_B. However, the first term, which involves the individual values of θ, is dependent upon the division of the total moment between the top and the bottom of the column. When the structure is quite regular, it is sufficiently accurate to assume that $M_A = M_B = \dfrac{M_t}{2}$. In an analysis of the Wilson and Maney bent this assumption was made, and the computed wind moments checked the true moments within a maximum error of 2 per cent. Nevertheless, since the use of fixed-end moments equal to the K-values of the columns is sufficiently accurate for a frame of reasonable regularity, a better approximation of the true division of the total moment between the two ends of the combined column seems desirable. At least, the refinement suggested subsequently will prove advantageous when the stiffness ratios between girders and columns at the two ends of a combined column vary from each other by more than 50 per cent.

The "stiffness ratio of girders to columns" can be defined as:

$$r = \frac{2\,\Sigma K_g \text{ of the girders between Stories } A \text{ and } B}{\Sigma K_c \text{ of the columns in Story } A + \Sigma K_c \text{ in Story } B}$$

This ratio will be called the r-ratio for any floor level.

Let $n_{top} = 2\, r_{top} + r_{bottom}$ and $n_{bottom} = 2\, r_{bottom} + r_{top}$. By checking the moments in several building frames, it has been found that the total column moments at the top and bottom of a story are to each other approximately as the n-values at the top and the bottom of the story. Accordingly, the total column moment is divided as follows:

$$M_{top} = M_t \frac{n_{top}}{n_{top} + n_{bottom}}, \text{ and } M_{bottom} = M_t \frac{n_{bottom}}{n_{top} + n_{bottom}}$$

Note that the n-value at the top of a basement column is ∞ and that the n-value at the bottom is $2\,\infty$. Hence, the total column moment would be divided in the ratio of 67% to the bottom and 33% to the top. This naturally assumes a fixed base at the bottom of the basement column. Economy usually requires that the moments at the top and the bottom of the basement columns correspond more closely than 2 to 1, and, accordingly, it seems desirable to place an upper limit of 60% on the division of moment.

By adding the total column moments above and below any floor level, the moment in the girder, M_g, is obtained. Since the K-value of the combined girder is known, its angle change or the joint rotation can be computed by area moments if the assumption is made that there is a point of contra-flexure at its mid-point. For this case, from Fig. 7,

$$\theta = \frac{M_g l}{6\,EI} = \frac{M_g}{6\,E\,2\Sigma K_g} = \frac{M_g}{12\,E\Sigma K_g} \qquad\qquad (5)$$

When all the θ-values have been determined, it is possible to compute the values of Δ for each story from the known relation,

$$\Delta = \theta_{\text{avg}}\,h + \frac{M_t h}{12\,E\,\Sigma K_c} \qquad\qquad (6)$$

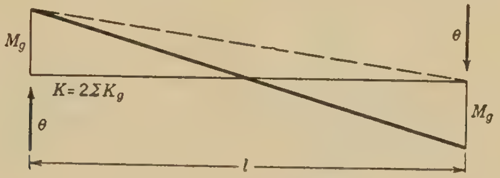


FIG. 7.—JOINT ROTATION.

The fixed-end moments in the individual columns caused by the side lurch are calculated from Equation (3), in which, K is successively the stiffness ratio for each individual column. There are no fixed-end moments in the girders. The fixed-end moments in the columns are balanced in the usual manner around the joints to obtain the first set of balanced moments.

When the balanced moments have been obtained, the column shears are calculated and added together. The total shear should equal the wind shear, and the difference is a measure of the error in the uncorrected moments. If these shears are within 10% of the true shears, the final moments can certainly be obtained with sufficient accuracy by proportion (maximum errors of about 6%). The final column moments are proportioned so as to make the calculated shears check the wind shears. The final girder moments are obtained by dividing the sum of the final column moments at a joint between the girders in the same ratio as that of the uncorrected girder moments.

If the first computed shears in certain stories vary from the true wind shears by more than 10 or 15%, the designer must use his judgment as to whether the foregoing method of correcting the moments by proportion will be sufficiently accurate to meet his requirements or whether it will be necessary to introduce correction moments into certain stories, as has been described in a preceding section. It seems improbable that the important moments obtained by proportion would ever be in error more than 10 or 12% when the calculated shears checked the true shears within 20 per cent. If continued experience with the method justifies this assumption, the need for introducing a set of correction moments probably will never arise.

The lower eight stories of the American Insurance Union Building were analyzed by the simplified method making use of the n-values as explained. The computations of the fixed-end moments for the lower eight stories are given in Table 3. The fixed-end moments are balanced, criterion ratios are computed, and final moments are determined in Fig. 8. Final moments are shown in parenthesis and are in units of inch-pounds divided by 100 000. These final moments are compared with those determined by the slope deflection

TABLE 3.—COMPUTATIONS FOR ESTIMATING STORY DEFLECTIONS, AMERICAN INSURANCE UNION BUILDING, ANALYSIS COMPLETED IN FIG. 8

Story	Height, in feet	ΣK_c, in inches³	ΣK_g, in inches³	r	n	Moment division, percentage	Shear, in kips	Total moment, M_t, in kip-feet	Total column moments, in kip-feet	$E\theta \div 1\,000$	Deflections, in inches			Fixed-end column moments, in inch-pounds ÷ 10 000		
											$E\theta_{avg} h \div 1\,000$	$E\delta \div 1\,000$	$E\Delta \div 1\,000$	A	B	C
			168.9	0.65						18.3						
6	10.33	261.0			1.90	51	300.0	3 100	1 580		2 330	1 470	3 800	643	743	1 012
					1.85	49			1 520							
			168.9	0.60						19.3						
5	11.00	300.8			1.75	51	307.4	3 380	1 730		2 570	1 480	4 050	746	848	1 173
					1.70	49			1 650							
			168.9	0.55						19.6						
4	10.58	312.6			1.67	50	315.0	3 330	1 660		2 460	1 350	3 810	760	864	1 193
					1.69	50			1 670							
			168.9	0.57						19.2						
3	11.82	280.0			3.12	41	322.8	3 820	1 570		1 820	1 940	3 760	600	681	942
					4.53	59			2 250							
			816.1	1.98						6.5						
2	15.25	544.0			4.37	60	332.3	5 070	3 040		2 160	1 710	3 870	796	986	1 670
					2.80	40			2 030							
			245.4	0.41						17.1						
M	12.96	640.2			1.27	49	342.1	4 430	2 170		2 880	1 070	3 950	1 128	1 393	2 360
					1.31	51			2 260							
			245.4	0.45						19.9						
1	18.58	446.2			3.50	40	352.9	6 560	2 620		3 050	3 270	6 320	876	1 083	1 838
					5.66	60			3 940							
			949.2	2.60						7.5						
B	22.15	285.0			∞	40	355.2	7 870	3 150		990	7 340	8 330	809	745	1 130
			∞	∞	2∞	60			4 720	0						

method in Part (*c*) of Table 2. The maximum variation in moment found by the simplified method when applied in this form was 9% when checked against the moments obtained by the slope deflection method. Note that this accuracy was obtained from balanced moments that had been carried out to two places only. It will be recalled that this same building was analyzed by the first form of the simplified method, in which fixed-end moments in proportion to the *K*-values of the columns are balanced, and that the maximum error in the final moments was 12 per cent. It will also be recalled that correction moments were introduced into one story in this analysis. Accordingly, the second form of the simplified method, which involves a preliminary estimate of story deflections, was more satisfactory for the analysis of this structure and will usually be found advantageous for the analysis of such irregular frames.

The time consumed by the two variations of the simplified method was not very different for this frame. In the first analysis, time was required for the introduction of correction moments into the frame, while in the second analysis about an equal amount of time was required to make the preliminary estimate of deflections. In either case, there is an appreciable saving over the time consumed by the general method of successive corrections, although the saving is not as important as was found for the more regular frame.

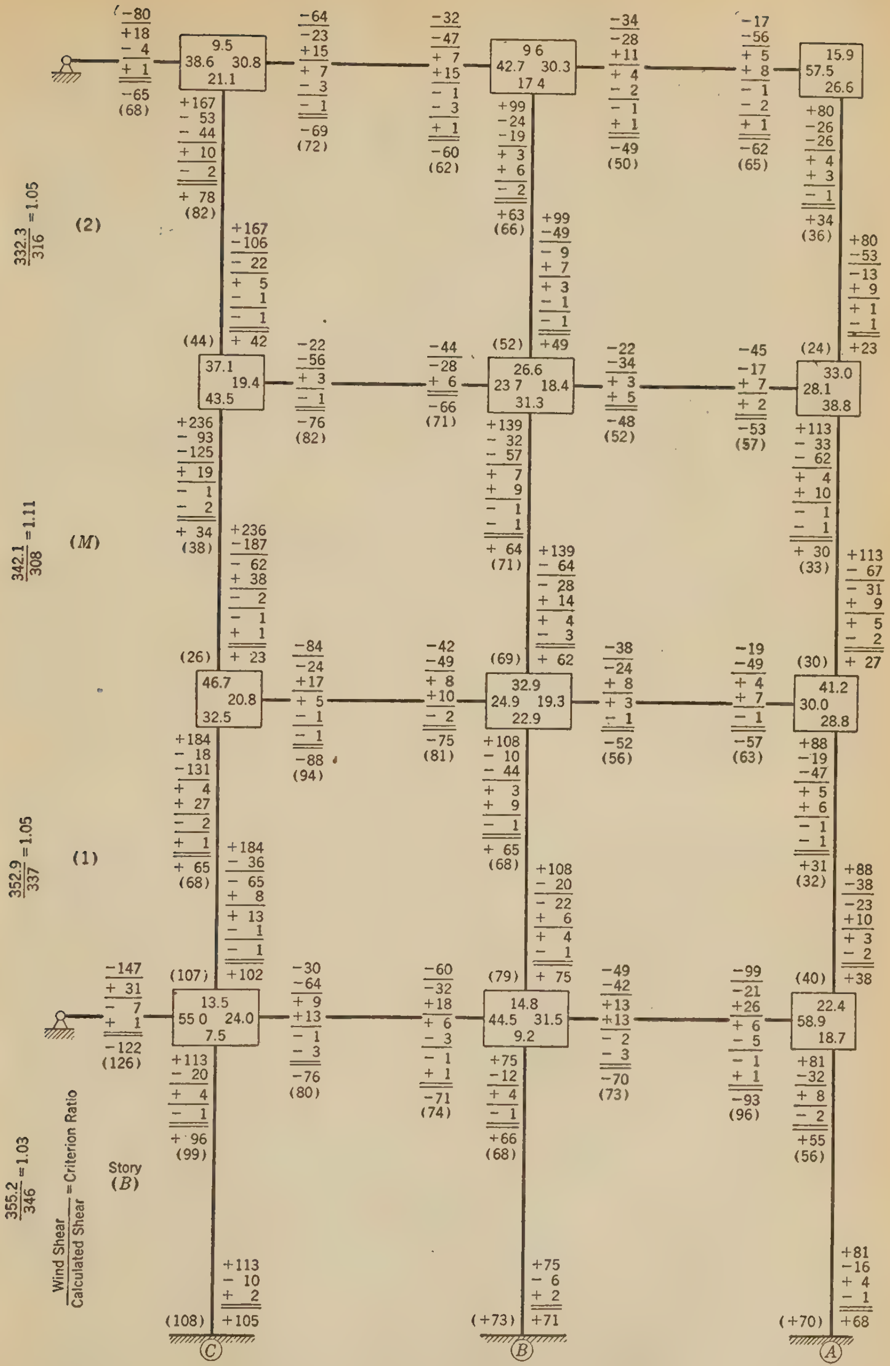

−80 +18 −4 +1 −65 (68)
9.5 38.6 30.8 21.1
−64 −23 +15 +7 −3 −1 −69 (72)
−32 −47 +7 +15 −1 −3 +1 −60 (62)
9.6 42.7 30.3 17.4
−34 −28 +11 +4 −2 −1 +1 −49 (50)
−17 −56 +5 +8 −1 −2 +1 −62 (65)
15.9 57.5 26.6
+167 −53 −44 +10 −2 +78 (82)
+99 −24 −19 +3 +6 −2 +63 (66)
+80 −26 −26 +4 +3 −1 +34 (36)
332.3/316 = 1.05
(2)
+167 −106 −22 +5 −1 −1 +42
(44)
+99 −49 −9 +7 +3 −1 −1 +49
(52)
+80 −53 −13 +9 +1 −1 +23
(24)
37.1 19.4 43.5
−22 −56 +3 −1 −76 (82)
−44 −28 +6 −66 (71)
26.6 23.7 18.4 31.3
−22 −34 +3 +5 −48 (52)
−45 −17 +7 +2 −53 (57)
33.0 28.1 38.8
+236 −93 −125 +19 −1 −2 +34 (38)
+139 −32 −57 +7 +9 −1 −1 +64 (71)
+113 −33 −62 +4 +10 −1 −1 +30 (33)
342.1/308 = 1.11
(M)
+236 −187 −62 +38 −2 −1 +1 +23
(26)
+139 −64 −28 +14 +4 −3 +62
(69)
+113 −67 −31 +9 +5 −2 +27
(30)
46.7 20.8 32.5
−84 −24 +17 +5 −1 −1 −88 (94)
−42 −49 +8 +10 −2 −75 (81)
32.9 24.9 19.3 22.9
−38 −24 +8 +3 −1 −52 (56)
−19 −49 +4 +7 −1 −57 (63)
41.2 30.0 28.8
+184 −18 −131 +4 +27 −2 +1 +65 (68)
+108 −10 −44 +3 +9 −1 +65 (68)
+88 −19 −47 +5 +6 −1 −1 +31 (32)
352.9/337 = 1.05
(1)
+184 −36 −65 +8 +13 −1 −1 +102
(107)
+108 −20 −22 +6 +4 −1 +75
(79)
+88 −38 −23 +10 +3 −2 +38
(40)
−147 +31 −7 +1 −122 (126)
13.5 55.0 24.0 7.5
−30 −64 +9 +13 −1 −3 −76 (80)
−60 −32 +18 +6 −3 −1 +1 −71 (74)
14.8 44.5 31.5 9.2
−49 −42 +13 +13 −2 −3 −70 (73)
−99 −21 +26 +6 −5 −1 +1 −93 (96)
22.4 58.9 18.7
+113 −20 +4 −1 +96 (99)
+75 −12 +4 −1 +66 (68)
+81 −32 +8 −2 +55 (56)
355.2/346 = 1.03
Story (B)
Wind Shear / Calculated Shear = Criterion Ratio
+113 −10 +2 +105
(108)
+75 −6 +2 +71
(+73)
+81 −16 +4 −1 +68
(+70)
C
B
A

FIG. 8.—BALANCING FIXED-END MOMENTS OBTAINED FROM THE DEFLECTIONS ESTIMATED IN TABLE 3, AMERICAN INSURANCE UNION BUILDING.

Effect of Column Deformation

Since the windward columns lengthen while the leeward columns shorten under wind load, there are fixed-end moments produced in the girders by this effect. The obvious procedure is to determine the direct stress deformations in the columns after having determined the wind moments. From these deformations the fixed-end girder moments are computed and balanced. These balanced moments must be made a part of the original analysis, and since they produce story shears, the criterion ratios will be changed. The final moments are these balanced moments plus the wind moments obtained by proportion using the new criterion ratios.

Naturally the direct stresses, if re-computed, would be found slightly changed, but a further revision of moments would seem to be unwarranted. In fact, it is only in the tower structure that column deformation becomes of importance in wind stress calculations. The writer has estimated the possible effect of column deformation in a discussion[7a] of the Second Progress Report of Sub-Committee No. 31, Committee on Steel of the Structural Division, on Wind-Bracing.

The effect of column deformation can be included directly in a second analysis whenever a revision of the sizes of the members makes a second analysis necessary. The approximate direct stresses in the columns are then known, and the fixed-end girder moments may be introduced and balanced along with the fixed-end column moments. Note, however, that the column moments must have been obtained from absolute rather than relative story deflections. Otherwise, the effect of column deformation would be either exaggerated or minimized when final moments are obtained by proportion.

The Open-Web or Vierendeel Truss

This structure is rather unusual, but does occur where clear openings are required in building design and has been used as a bridge structure, largely in Europe. Its analysis for fixed loads may be made by the simplified method either in the form of balancing fixed-end moments in proportion to the K-values of the members or by balancing fixed-end moments obtained from the estimated panel deflections.

The analysis of the open-web truss by balancing fixed-end moments equal to the K-values of the members is almost identical to the similar process for a building frame. In this structure the fixed-end moments occur only in the horizontal members or chords, provided the effect of direct stress deformation is to be neglected, as is usual. Depending upon the number of panels in the truss, it may or may not be necessary to introduce correction moments into the various panels before adjusting moments by proportion. The criterion ratios for adjacent panels must be made to correspond within about twice the maximum variation to be expected between the calculated moments and the true moments. In other words, if a 5 or 6% error is permissible in the final moments, a 10% variation may be allowed between the criterion ratios for adjacent panels.

[7a] *Proceedings*, Am. Soc. C. E., May, 1932, p. 956.

It will be an obvious improvement to assume that a parabolic or circular deflection curve exists rather than one of triangular shape. The latter is assumed automatically to be the deflection curve if all fixed-end moments are taken as equal to the K-values of the chords. A parabola will serve as well as any other standard curve, and its properties are most readily determined. From the relative assumed deflections of the panel points, one determines the ratios for the various panels by which the K-values of the chords must be multiplied to obtain the original fixed-end moments. The process is simple enough to make a detailed explanation seem unnecessary.

USE OF THE SIMPLIFIED METHOD OF WIND STRESS ANALYSIS IN THE DESIGN OF A TALL BUILDING FRAME

The simplified method of wind stress analysis is convenient for use in the design of a tall building frame. Any statically determinate method of analysis with which the designer is familiar may be used in estimating the wind moments and direct stresses to be used in a preliminary design of the structure. The statically determinate method of analysis must be supplemented by the designer's judgment wherever there are extreme irregularities in the frame. When the sizes of the members have been determined for the most serious combination of dead load, live load, and wind, the simplified method of analysis should be used to determine the true wind moments in this frame.

Slight revisions of the proportions of the frame will probably be necessary to care for the new distribution of wind moments. If the preliminary design was made by an experienced designer, these changes will usually be so unimportant that a new analysis will not be required. Where considerable change is found necessary in the sizes of a few members, it may be desirable to re-distribute the moments about the joints into which these members frame. If radical changes must be made in the sizes of a number of members, it will be necessary to repeat the analysis. However, in this second analysis, the designer has at his command an almost exact knowledge of the story deflections (obtainable from the data of the first analysis), and, therefore, the labor of the second analysis is slightly reduced.

This outline of design procedure makes it perfectly clear that the experienced designer has a real advantage over the beginner in the use of this method. The design of any indeterminate structure should be simplified by a previous knowledge of correct proportions. Any method of design in which preliminary proportions are set essentially by rule of thumb does not take full advantage of the reduction of effort that experience should offer.

Naturally, the question of economy enters into the problem of establishing the proportions of the structure. Minimum wind moments will occur when points of contra-flexure exist at the centers of the members, and the smallest total direct stresses in the columns occur when the direct stresses in the interior columns are zero. However, a wind stress of from 25 to 50% of the dead load and live load stress in a member is usually allowed without requiring an increase in the size of the member. Accordingly, it is not necessarily true that there will be an important economy obtained by forcing the points of contra-flexure to the centers of the members or by proportioning the structure so that there will be zero direct stresses in the interior columns. In fact, the opposite may readily be true.

Near a point of extreme irregularity in the structure, the designer should study the matter of proportions carefully. For instance, when the lower story columns are fixed at their bases, it is frequently considered desirable to use heavy girders at the level of the first floor to force the points of contra-flexure in the lower story columns down near their mid-height. The importance of this arrangement increases with increase in the height of the building. However, one should realize that a heavy girder at the first floor will also require a reasonably heavy girder at the second floor, neither of which can be stressed up to its allowable stress. This heavy second floor girder is required in order to keep the points of contra-flexure in the second-story columns near their mid-height. The question arises as to whether or not it is more economical to place excess material in these first and second-floor girders rather than to increase the sizes of the first-story columns to care for heavy wind moments. This example is typical of the kind of problem that the designer must investigate around each point of extreme irregularity in the frame. A knowledge of the simplified method is extremely valuable in this connection.

Conclusion

This paper has been limited to a presentation of a general system of analysis for stresses in continuous frames that undergo joint translations but in which the loading is fixed. An analysis for the effect of moving loads is an entirely different matter and is not considered. The reader will probably see such an application in the calculation of the maximum stresses for a Vierendeel bridge truss. Since the value of this method of analysis is dependent upon its usefulness to the designer, the writer has indicated briefly its application to the design of the most important structure in this field—the tall building frame. A discussion of the proper proportions of building frames was intentionally omitted from the paper, although a study of the relative economy of frames having various deflection curves would be both interesting and valuable.

Since certain examples mentioned in this paper have been solved to within a range of error of 2 or 3%, it is feared that the reader may draw the conclusion that this precision is put forward as desirable. Such precision is clearly wasted because even the physical properties of materials are not known to this degree of exactness. On the other hand, it seems to the writer that the entire attitude of the designer must be demoralized by the knowledge that his tool for wind-stress analysis may be in error by from 50 to 100 per cent. Careless design would be the natural result. Any tool for wind stress analysis, such as the simplified method, by which the designer can compute the wind stresses to conform to a predetermined standard of accuracy, should tend to increase his confidence in the value of his calculations and improve the quality of his design work. Compared with the importance of this result, the value of a discussion as to whether the standard of accuracy in wind stress analysis should be set at a maximum error of 5, 10, or 20% seems trivial.

DISCUSSION

FRANCIS P. WITMER,[8] M. AM. SOC. C. E. (by letter). — Apparently, by making a reasonably close preliminary estimate of the horizontally deflected positions of the various joints, it is expected that the effect of "side-sway" may be found more expeditiously by Professor Grinter's method than by the process of repeatedly equalizing shears in each story, as described in the Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings.[9]

Apparently, the effect of vertical translation of joints, due to column distortions under direct wind stress, may be found either at the same time with the effect of horizontal displacements by making this consideration a part of the original analysis, or it may be included in a second analysis covering only these vertical translations.

If the analysis of the frame is first made by the method described in the progress report previously mentioned, neglecting column axial distortions, the horizontal movements will be fully cared for. A second analysis by the Cross method may then be performed for the vertical distortions only, considering these as producing secondary moments, and following Professor Grinter's method for obtaining the starting fixed-end moments in cross-girders. Should this second analysis result in new column axial distortions of sufficient size to justify it, a third analysis, and even a fourth, may be necessary to determine with sufficient accuracy the moments resulting from vertical displacements. The combination of these secondary moments (and, if required, those also of higher orders) with moments from the original analysis will produce the final moments in the frame.

Professor Grinter describes a solution of secondary stresses in a truss based upon his method, using a Williot diagram to obtain the translations of joints from which starting fixed-end moments are found. The writer described[10] a similar method in 1932.

ELMER K. TIMBY,[11] ASSOC. M. AM. SOC. C. E. (by letter).—In this paper Professor Grinter treats wind stresses in building frames of both regular and irregular proportions. To develop the methods presented, certain assumptions were made, and the purpose of this discussion is to inquire as to whether variations from these primary assumptions materially affect the reliability of the results obtained when using the simplified method.

The first assumption made in the treatment of regular symmetrical frames is that the deflections in all stories are in proportion to the story heights, or, mathematically, that $\frac{\Delta}{h}$ is a constant.

In the development of the method for estimating deflections of irregular building frames Professor Grinter assumes that the total moment in any column other than the basement column is equally divided between the two ends. This is equivalent to assuming a point of contra-flexure at mid-story height. For the basement column, the end moments are assumed to be 40% of the total at the top and 60% of the total at the bottom, which slightly raises the point of contra-flexure.

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[8] Director, Dept. of Civ. Eng., Univ. of Pennsylvania, Philadelphia, Pa.

[9] *Proceedings*, Am. Soc. C. E., February, 1932, p. 226, Step 4.

[10] *Engineering News-Record*, January 28, 1932.

[11] Instr., Eng., School of Eng., Princeton Univ., Princeton, N. J.

It is stated in the paper that a wind bent of ideal proportions should have identical columns in each story, and stories of constant height; and, further, that the K-values of the columns and girders should increase from the top downward in direct proportion to the story shears.

In 1930, Messrs. H. W. Coultas and V. H. Lawton, using a Beggs deformeter apparatus, conducted a most complete experimental analysis of a building frame.[12] The writer has calculated by the area-moment method the deflections of this structure, using the moments determined by the experimental analysis.

It will be seen from Fig. 9(a) that the frame is six stories high, totaling 79 ft to the roof beams, and that its width of 51 ft is equally divided into two bays, making it symmetrical about the center column, $C(2)$. The wind load was assumed to have an intensity of W lb per unit of area above the point, A, shown slightly above the center of the left column, and three-fourths of W lb below Point A. The shaded areas on Fig. 9(a) represent the moments of inertia of the members. It will be observed that the values for the columns increase from top to bottom quite uniformly; that the story heights are nearly equal, having a maximum variation of 1 ft; and, further, that the frame is regular and symmetrical. Therefore, this frame selected at random satisfies to a remarkable degree the conditions stipulated by Professor Grinter as defining an ideal frame.

Fig. 9(b) is a shaded moment diagram for the center column, $C(2)$. It will be observed that the columns in the top three stories have no point of zero moment and, therefore, no point of contra-flexure, and that the second-story column has the point of contra-flexure extremely close to the lower end. Both these observations appear to be at variance with the assumptions made by Professor Grinter.

Fig. 9(c) gives the deflection of the center column, $C(2)$, from a tangent at the base. It will be seen that the deflections in the stories are not proportional to the story heights. If this were true, the deflection curve would be a straight line from top to bottom.

The variation of the slope of the deflection curve may be observed in Fig. 9(d). These values of $\frac{\Delta}{h}$, which are assumed to be constant in Professor Grinter's paper, change from $+136$ to -4. These values were computed, assuming the tangent at the base to be vertical.

Any method which reduces the amount of time or labor required for the solution of a problem, and which, at the same time, yields a high degree of accuracy, is to be highly commended. The writer would like to inquire, however, whether or not these apparently radical differences between assumptions and conditions materially affect the accuracy obtainable with the author's simplified method.

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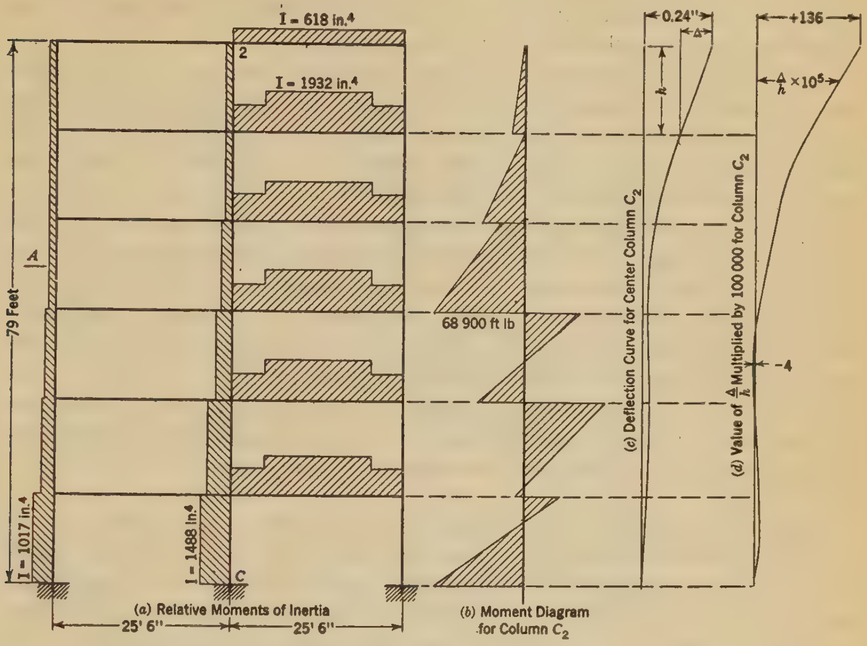
[12] *The Structural Engineer* (London), October, 1930.



(a) Relative Moments of Inertia

(b) Moment Diagram for Column C_2

(c) Deflection Curve for Center Column C_2

(d) Value of $\frac{\Delta}{h}$ Multiplied by 100 000 for Column C_2

FIG. 9.—FRAME ANALYZED BY MESSRS. H. W. COULTAS AND V. H. LAWTON.

The writer felt, before making this investigation, that the assumption, $\frac{\Delta}{h}$ = a constant, might not be reliable for frames of great height, because of the important effect of column strains. He was surprised, however, to find that the assumption does not appear to be valid for some simple regular frames of low height. A discussion of this particular point by Professor Grinter would be appreciated.

N. A. RICHARDS,[13] M. AM. SOC. C. E. (by letter).—In this paper Professor Grinter offers a method for the analysis of continuous frames in which joint displacements occur. After illustrating the applicability of this method to other frames, he shows how it may be used in the design of high building frames subject to wind. Like other methods of analysis, it presupposes the existence of a design to analyze, and this design must be prepared by some method wholly distinct from this method of analysis.

In buildings of moderate height and dimensions, in which the bracing is not the prime feature of the design, it might often happen that this method would prove useful in a check to determine whether the ordinary members in the building frame would be overstressed from an assumed wind. However, if the building frame presents a major wind problem, due to its height or narrowness, it seems to the writer that it is approaching the design in a backhanded way to create it by some approximate method, and then analyze and revise it by a method such as is suggested.

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[13] Vice-Pres., Purdy & Henderson Co., New York, N. Y.

In the design of structural members generally, such as girders, beams, and columns, subject to dead and live loads, the designer proceeds in a direct manner to the selection of members sufficient for the conditions of loading, bending, and shear, and without the necessity of a review of some other method to determine whether the members will act as designed.

In their Third Progress Report the members of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings indicate[14] their belief that they have suggested a method of approach to wind-bracing design, which enables the designer to proceed in as direct a manner to the selection of the members of such a frame, and with no more necessity for review by another analytical method.

It is not the purpose of this comment to criticize the merits of Professor Grinter's method, but rather to bring out the fact that it is of value principally as a method of analysis, and not as a tool of design.

JOHN B. LETHERBURY,[15] JUN. AM. SOC. C. E. (by letter).—The author's method of determining wind moments and deflections supplies a rapid means of solving an otherwise laborious problem. The writer was particularly interested in the conception of a combined girder and column replacing the wind bent, and he desired to experiment with the method on a frame with which he is familiar.

The frame shown in Fig. 10(a) was chosen originally for use in comparing a statically determinate method with the method of moment and shear adjustment suggested[16] by Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings. The only requirements were, that it should be somewhat irregular, and that its stiffness should increase from top to bottom. Moments were obtained with an accuracy of about 10% by three cycles of distribution.

The deflections of Columns A and D were calculated by the method suggested by the Sub-Committee[16] and are shown in Fig. 10(b). The deflections of the combined column and girder calculated by Professor Grinter's method are also shown. All three of the curves approximate a straight line. The combined column and girder deflects but little more than either of the columns, the maximum difference being about 7 per cent. This would seem to indicate that the fixed-end moments could be considered as proportional to the stiffness of the columns.

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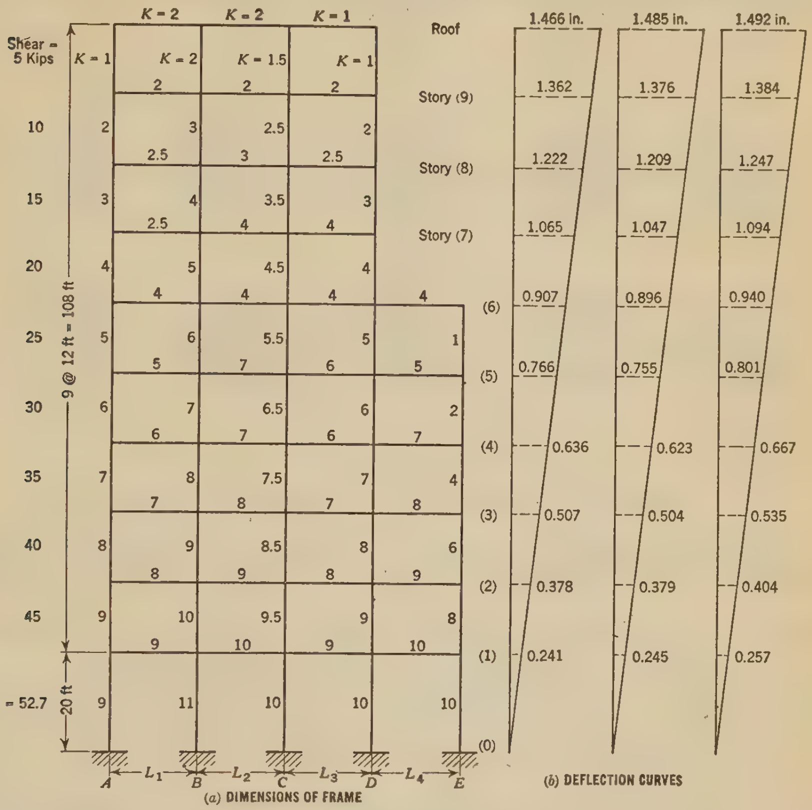
[14] *Proceedings*, Am. Soc. C. E., December, 1933, p. 1601.

[15] Instr., Civ. Eng., Drexel Inst., Philadelphia, Pa.

[16] Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings, *Proceedings*, Am. Soc. C. E., February, 1932, p. 213.

A slight break occurs in the deflection curves at the top of the fifth story, indicating that the effect of the set-back occurs at that point. An increase in uniform wind loading on the upper stories would cause a similar break in the curve which would then approximate two straight lines joined by a curved line. A much higher frame might show a deflection curve more nearly approximating a flat parabola than a straight line, but as Professor Grinter has pointed out, the story deflections would be sufficiently uniform to warrant the assumption of fixed-end moments proportional to the K-values of the columns.


K = 2
K = 2
K = 1
Shear = 5 Kips
K = 1
K = 2
K = 1.5
K = 1
Roof
Story (9)
Story (8)
Story (7)
(6)
(5)
(4)
(3)
(2)
(1)
(0)
10
15
20
25
30
35
40
45
= 52.7
9 @ 12 ft = 108 ft
20 ft
A
B
C
D
E
L_1
L_2
L_3
L_4
1.466 in.
1.362
1.222
1.065
0.907
0.766
0.636
0.507
0.378
0.241
1.485 in.
1.376
1.209
1.047
0.896
0.755
0.623
0.504
0.379
0.245
1.492 in.
1.384
1.247
1.094
0.940
0.801
0.667
0.535
0.404
0.257

(a) DIMENSIONS OF FRAME

(b) DEFLECTION CURVES

FIG. 10.

In Fig. 11, the moments, in foot-kips, are shown for the set-back portion of the frame. The moment in each case faces the member to which it is applied. Values in parenthesis—thus, (10.00)—are by the Grinter method; those marked by an asterisk—thus, 10.00*—are by the Bowman method; and the remainder are by three cycles of moment and shear distribution. Fixed-end moments were calculated from the story deflections and carried through two cycles of distribution, which brought the final moments into close agreement with those obtained by three cycles of moment and shear distribution. The criterion ratio for the sixth story was 1.05.

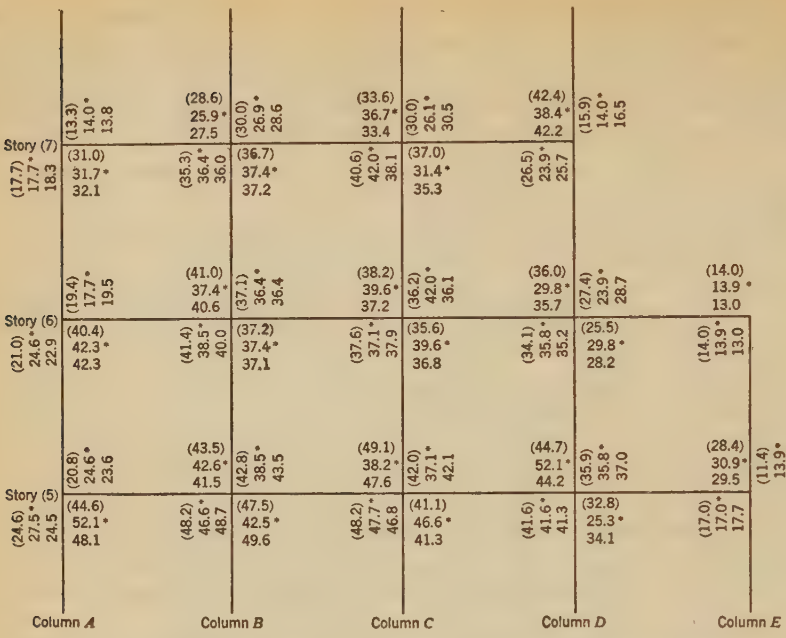

(13.3) 14.0* 13.8
(28.6) 25.9* 27.5
(30.0) 26.9* 28.6
(33.6) 36.7* 33.4
(30.0) 26.1* 30.5
(42.4) 38.4* 42.2
(15.9) 14.0* 16.5
Story (7)
(17.7) 17.7* 18.3
(31.0) 31.7* 32.1
(35.3) 36.4* 36.0
(36.7) 37.4* 37.2
(40.6) 42.0* 38.1
(37.0) 31.4* 35.3
(26.5) 23.9* 25.7
(19.4) 17.7* 19.5
(41.0) 37.4* 40.6
(37.1) 36.4* 36.4
(38.2) 39.6* 37.2
(36.2) 42.0* 36.1
(36.0) 29.8* 35.7
(27.4) 23.9* 28.7
(14.0) 13.9* 13.0
Story (6)
(21.0) 24.6* 22.9
(40.4) 42.3* 42.3
(41.4) 38.5* 40.0
(37.2) 37.4* 37.1
(37.6) 37.1* 37.9
(35.6) 39.6* 36.8
(34.1) 35.8* 35.2
(25.5) 29.8* 28.2
(14.0) 13.9* 13.0
(20.8) 24.6* 23.6
(43.5) 42.6* 41.5
(42.8) 38.5* 43.5
(49.1) 38.2* 47.6
(42.0) 37.1* 42.1
(44.7) 52.1* 44.2
(35.9) 35.8* 37.0
(28.4) 30.9* 29.5
(11.4) 13.9* 11.8
Story (5)
(24.6) 27.5* 24.5
(44.6) 52.1* 48.1
(48.2) 46.6* 48.7
(47.5) 42.5* 49.6
(48.2) 47.7* 46.8
(41.1) 46.6* 41.3
(41.6) 41.6* 41.3
(32.8) 25.3* 34.1
(17.0) 17.0* 17.7
Column A
Column B
Column C
Column D
Column E

FIG. 11.—MOMENTS IN SET-BACK PORTION COMPARED BY THREE METHODS.

TABLE 4.—COMPARISON OF COLUMN SHEAR, IN KIPS, CALCULATED BY THE STATICALLY DETERMINATE AND THE SIMPLIFIED METHODS

Story	COLUMN A		COLUMN B		COLUMN C		COLUMN D		COLUMN E		Maximum difference in percentage of simplified method
	Statically determinate method	Simplified method	Statically determinate method	Simplified method	Statically determinate method	Simplified method	Statically determinate method	Simplified method	Statically determinate method	Simplified method	
10	0.98	0.83	1.96	1.91	1.46	1.48	0.61	0.77	.....	.....	21
9	1.78	1.87	3.29	3.32	3.16	3.09	1.78	1.77	.....	.....	5
8	2.59	2.51	4.98	4.85	4.84	5.06	2.59	2.58	.....	.....	4
7	2.94	3.19	6.07	6.03	7.00	6.40	3.99	4.32	.....	.....	9
6	4.10	3.48	6.41	7.02	6.19	6.65	5.97	5.83	2.32	2.12	15
5	4.59	4.22	7.77	7.85	7.95	7.85	6.94	7.07	2.84	2.79	9
4	5.34	5.00	8.60	8.60	8.40	8.45	8.17	8.15	4.50	4.80	7
3	6.04	5.60	9.64	9.80	9.44	9.50	9.24	9.70	5.63	5.60	8
2	6.73	6.00	10.7	11.0	10.5	10.7	10.3	11.0	6.74	6.14	12
1	8.61	8.30	12.1	12.2	11.3	11.1	11.3	11.4	9.46	9.50	4

The statically determinate method[17] used is shown for comparison. It was devised by H. L. Bowman, M. Am. Soc. C. E. It has been found to give reasonably good results on a number of frames. This is due largely to the excellent shear division between columns obtained by an empirical equation based on the study of several bents which were solved by more exact methods. The other two assumptions necessary for solution are those locating points of contra-flexure in the columns and girders. The writer mentions this method because of its good results and its flexibility in application to design. Table 4 gives a comparison of the column shears, in kips, in the frame shown.

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[17] "Structural Theory," by Hale Sutherland and Harry Lake Bowman, Members, Am. Soc. C. E.

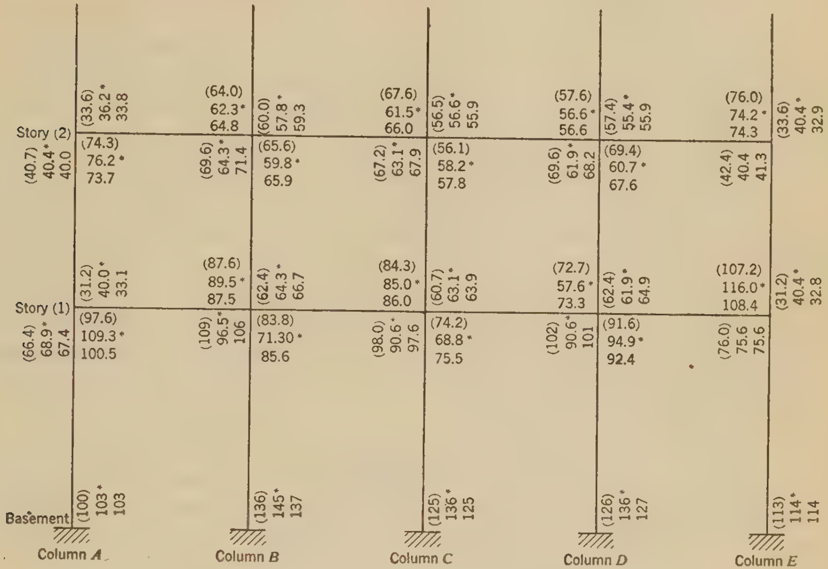
Story (2)
Story (1)
Basement
(33.6) 36.2* 33.8
(64.0) 62.3* 64.8
(60.0) 57.8* 59.3
(67.6) 61.5* 66.0
(56.5) 56.6* 55.9
(57.6) 56.6* 56.6
(57.4) 55.4* 55.9
(76.0) 74.2* 74.3
(33.6) 40.4* 32.9
(40.7) 40.4* 40.0
(74.3) 76.2* 73.7
(69.6) 64.3* 71.4
(65.6) 59.8* 65.9
(67.2) 63.1* 67.9
(56.1) 58.2* 57.8
(69.6) 61.9* 68.2
(69.4) 60.7* 67.6
(42.4) 40.4 41.3
(31.2) 40.0* 33.1
(87.6) 89.5* 87.5
(62.4) 64.3* 66.7
(84.3) 85.0* 86.0
(60.7) 63.1* 63.9
(72.7) 57.6* 73.3
(62.4) 61.9* 64.9
(107.2) 116.0* 108.4
(31.2) 40.4* 32.8
(66.4) 68.9* 67.4
(97.6) 109.3* 100.5
(109) 96.5* 106
(83.8) 71.30* 85.6
(98.0) 90.6* 97.6
(74.2) 68.8* 75.5
(102) 90.6* 101
(91.6) 94.9* 92.4
(76.0) 75.6 75.6
(100) 103* 103
(136) 145* 137
(125) 136* 125
(126) 136* 127
(113) 114* 114
Column A
Column B
Column C
Column D
Column E

FIG. 12.—COMPARISON OF MOMENTS IN THE TWO LOWER STORIES.

In Fig. 12, the comparison for the two lower stories is given, the units and symbols being the same as those in Fig. 11. In this, the fixed-end moments calculated by the simplified method were carried through four cycles to a criterion ratio of 0.96. The statically determinate method is within 20% of agreement.

The writer believes that the simplified method reduces the labor of wind-stress analysis and the calculations of frame deflections to a minimum consistent with reasonable accuracy. The conception of a combined column and girder may be developed into a yardstick for the measurement of tower deflection and vibration. The natural period of the frame could be calculated from its mass and the deflections of the combined column and girder, assuming harmonic motion and some possible gust forces. The amplitudes and frequencies thus obtained would be in excess of the actual ones in the finished building. It seems logical to attack the problem by means of Professor Grinter's conception.

FREDERICK MARTIN WEISS,[18] JUN. AM. SOC. C. E. (by letter).—The author is to be commended for his efforts to simplify a solution to one of the most complex problems arising in structural engineering. Although the system of analysis presented may deserve considerable space in college textbooks and discussion in academic circles, it is scarcely likely to prove to be the "short-cut" long awaited by practising structural engineers. Conclusions as to its practical worth must not be drawn from the sample calculations relating to the lower parts of the Wilson and Maney bent and the American Insurance Union Building bent.

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[18] Passaic, N. J. Mr. Weiss resigned from the Society on December 28, 1933.

In the first place, nothing but the simplest method of wind-stress analysis can possibly be applied to a tall building comprising a vast number of members requiring such attention. The three sample calculations presented by Professor Grinter, contain 14, 43, and 21 members, respectively, and from these an estimate may be obtained of the time and labor required per member. Then, when one is informed that such a skyscraper as the 56-story Irving Trust Company Building, in New York City, contains approximately 6 200 members in the wind-resisting part of the frame, it becomes evident that, because of the limited time allowed a structural engineer for the complete design of a steel-framed building, "exact" or "semi-exact" analyses must be abandoned and resort made to less tedious and laborious methods. This is not a plea for the adoption of quick and inaccurate wind-stress calculations, but merely a revelation of the magnitude of the task confronting designers of high buildings.

In the second place, Professor Grinter seems to have overlooked one of the peculiar characteristics of modern tall buildings. Unfortunately, these characteristics do not occur in the Wilson and Maney bent, or in the American Insurance Union Building bent, which were necessarily chosen because slope-

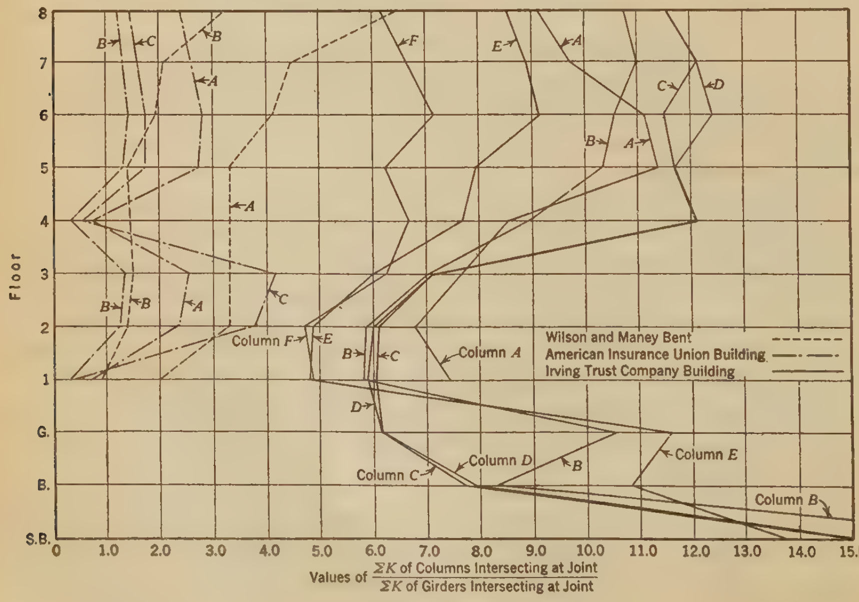

FIG. 13

deflection analyses were available for them. In general, it will be found that in tall buildings the stiffness ratios of the lower-story columns will be many times the stiffness ratios of the girders connected to them. In applying the simplified method of successive corrections to the analyses of such building bents, the convergence of approximations will be very slow because, upon balancing any joint, very little of the unbalanced moment will be distributed to the girders. Since the original fixed-end moments of the girders are zero, and since the balancing moments are small and the carry-over moments are even smaller, it is evident that a great many approximations must be performed before the final balanced moments of the girders at any joint are sufficiently large to equal the sum of the column moments at the same joint.

A rough estimate of the rapidity with which convergence of the approximations may be expected can be obtained from the quotient of the sum of the stiffness ratios of the columns intersecting at a joint divided by the sum of the stiffness ratios of the girders intersecting at the same joint. The larger these quotients are, the slower will be the rate of convergence. In Fig. 13 values of these factors are plotted for the lower eight floors of the Wilson and Maney bent and for the lower eight floors of the American Insurance Union Building bent, together with values for the lower eleven floors of a bent selected through the tower of the Irving Trust Company Building. The necessary dimensions, in feet, and moments of inertia, in inches⁴, for the Irving Trust Company Building are arranged for reference in Table 5.

TABLE 5.—DIMENSIONS AND CHARACTERISTICS OF THE IRVING TRUST COMPANY BUILDING BELOW THE NINTH FLOOR

Story	Columns							Girders				
	Height, in feet	Moments of Inertia, in Inches⁴										
(1)	(2)	*A* (3)	*B* (4)	*C* (5)	*D* (6)	*E* (7)	*F* (8)	*A–B* (span, 26.08 ft) (9)	*B–C* (span, 15.83 ft) (10)	*C–D* (span, 27.00 ft) (11)	*D–E* (span, 15.83 ft) (12)	*E–F* (span, 19.58 ft) (13)
	12.5	5 609	15 969	15 314	15 314	13 317	4 476					
8								2 735	2 184	2 234	2 184	2 457
	12.5	6 421	16 717	16 717	16 717	14 600	5 221					
7								2 735	2 184	2 234	2 184	2 457
	12.5	6 421	16 717	16 717	16 717	14 600	5 221					
6								2 735	2 457	2 234	2 184	2 457
	12.5	8 238	17 501	17 501	17 501	15 314	6 011					
5								3 021	2 457	2 234	2 457	3 021
	12.5	8 238	17 501	17 501	17 501	15 314	6 011					
4								4 380	2 457	2 234	2 457	3 344
	12.5	9 785	18 518	18 518	18 518	15 969	8 238					
3								4 721	3 021	4 721	3 021	3 670
	16.21	9 785	18 518	18 518	18 518	15 969	8 238					
2								6 050	3 217	6 050	3 217	4 985
	14.00	13 621	19 894	20 374	19 894	17 379	9 785					
1								6 050	3 217	6 050	3 217	4 985
	18.00	13 621	19 894	20 374	19 894	17 379	9 785					
*G**								4 045	1 543	9 306	1 543	1 954
	14.00	†	21 814	22 774	22 774	18 518	†					
*B**								6 664	2 457	8 301	2 184	2 457
	12.00	†	21 814	22 774	22 774	18 518	†					
*SB**								1 954	1 954	3 596	1 954	1 954
	12.33	†	22 774	23 318	23 318	18 998	†					

* *G* = ground floor; *B* = basement; and *SB* = Sub-basement. † Foundation walls.

It will be noticed in Fig. 13 that the values for the Wilson and Maney bent and for the American Insurance Union Building bent are very much smaller, in general, than the values for the Irving Trust Company Building.

In conclusion, therefore, it may be stated that to attain a desired degree of accuracy, the practical application of this variation of the Cross method may become such a Herculean undertaking as to overshadow completely the task performed by Professors Wilson and Maney in solving sixty simultaneous equations.[19]

Raymond C. Reese,[20] Assoc. M. Am. Soc. C. E. (by letter).—In applying Professor Cross' method of moment distribution and Professor Grinter's simplification, the designer is mainly interested in how quickly he can obtain reasonably accurate results. From the nature of the problem, no "exact" solution can be expected; nor is it necessary.

The work required is dependent upon the speed with which the moment distribution converges; while the accuracy can only be determined by agreement with other recognized methods of attack. The speed of convergence, again, depends upon the relative stiffness of the members, particularly the ratio of story-column stiffness to the stiffness of one tier of beams.

Most of the recorded examples are fairly regular frames, as that is the only kind amenable to the so-called "exact" methods that are used as a check. In actual work, however, it is seldom possible to make such simple and regular frames. This is particularly true of reinforced concrete buildings, where the designer has an unlimited choice of sections.

The writer was making a review of the wind stresses in an ordinary fifteen-story reinforced concrete hotel building for another purpose, and it seemed an excellent chance to determine the speed of convergence of the distribution. The remainder of this discussion will be concerned solely with this one phase of the problem.

Fig. 14(*a*) is a diagrammatic elevation of the end bent of this building showing dimensions. The carry-over factors and stiffness ratios for this bent are presented in Table 6. The building is sufficiently high and narrow to make the wind stresses a factor in the design. In the original design Fleming's method was used, of taking points of inflection at mid-girder span and mid-column height, and apportioning column shears on the basis of one-half portion to the exterior columns and a full portion to the interior columns.

The size and shape of many of the members were determined by architectural considerations. The column adjoining the stairs was of long, narrow cross-section, to clear the stair shaft, and was built with three spirals. The depth of spandrels was fixed by the legal height of the window heads. They could not be inverted because of the steam return lines buried in the walls; but were deepened at the ends to afford as much wind resistance as possible. Double reinforcement was used in these beams.

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[19] *Bulletin No. 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[20] Secy. and Chf. Engr., The Hausman Steel Co., Toledo, Ohio.

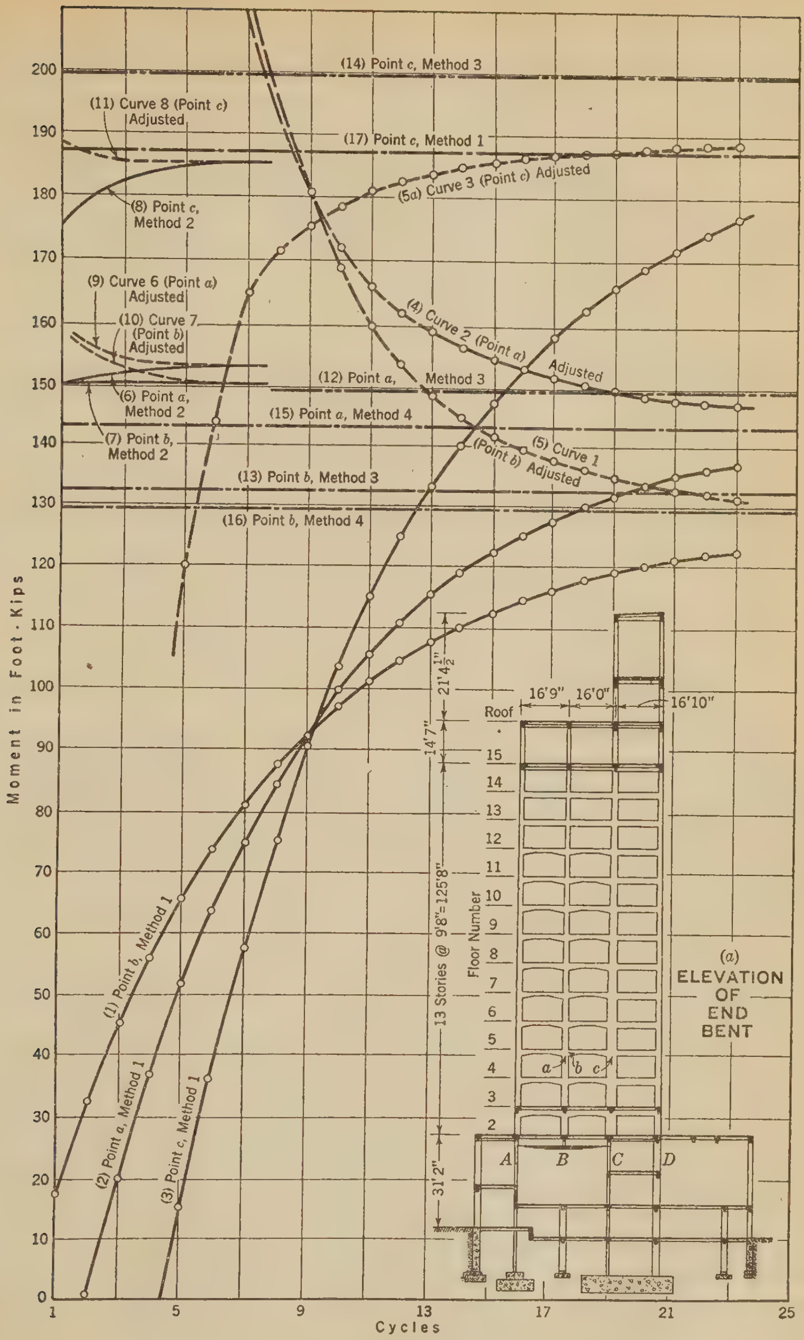
(14) Point c, Method 3
(11) Curve 8 (Point c) Adjusted
(17) Point c, Method 1
(5a) Curve 3 (Point c) Adjusted
(8) Point c, Method 2
(9) Curve 6 (Point a) Adjusted
(10) Curve 7 (Point b) Adjusted
(4) Curve 2 (Point a) Adjusted
(12) Point a, Method 3
(6) Point a, Method 2
(15) Point a, Method 4
(7) Point b, Method 2
(5) Curve 1 (Point b) Adjusted
(13) Point b, Method 3
(16) Point b, Method 4
(1) Point b, Method 1
(2) Point a, Method 1
(3) Point c, Method 1
Moment in Foot-Kips
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
1
5
9
13
17
21
25
Cycles
21'4½"
14'7"
13 Stories @ 9'8"=125'8"
31'2"
Floor Number
Roof
15
14
13
12
11
10
9
8
7
6
5
4
3
2
16'9"
16'0"
16'10"
a
b
c
A
B
C
D
(a) ELEVATION OF END BENT

FIG. 14

TABLE 6.—STIFFNESS, K, AND CARRY-OVER FACTORS, REINFORCED CONCRETE BUILDING

Item No.	Floor	COLUMNS (CARRY-OVER FACTOR, 0.5)				GIRDERS					
		A	*B*	*C*	*D*	*AB*		*BC*		*CD*	
		Stiffness, *K*	Stiffness, *K*	Stiffness, *K*	Stiffness, *K*	Stiffness, *K*	Carry-over	Stiffness, *K*	Carry-over	Stiffness, *K*	Carry-over
1........	Roof					0.41	0.5	0.41	0.5	0.97	0.5
2........		0.32	0.32	0.95	0.95						
3........	15					0.41	0.5	0.41	0.5	0.97	0.5
4........		0.32	0.32	2.10	1.52						
5........	14					0.41	0.5	0.41	0.5	0.97	0.5
6........		0.32	0 83	4.60	1.61						
7........	13					0.41	0.5	0.41	0.5	0.97	0.5
8........		0.56	1.52	7.80	2.32						
9........	12					0.65	0.45	0.65	0.45	2.00	0.5
10........		0.95	2.32	9.00	2.56						
11........	11					0.65	0.45	0.65	0.45	2.00	0.5
12........		1.52	3.61	9.00	3.61						
13........	10					0.65	0.45	0.65	0.45	2.00	0.5
14........		2.10	4.43	9.00	3.61						
15........	9					0.65	0.45	0.65	0.45	2.00	0.5
16........		2.32	5.14	9.00	3.61						
17........	8					0.65	0.45	0.65	0.45	2.00	0.5
18........		3.61	5.14	9.00	3.61						
19........	7					1.00	0.667	1.00	0.667	2.00	0.5
20........		4.71	5.14	26.60	3.61						
21........	6					1.00	0.667	1.00	0.667	2.00	0.5
22........		5.14	5.14	26.60	4.32						
23........	5					1.00	0.667	1.00	0.667	2.00	0.5
24........		5.14	7.06	26.60	4.32						
25........	4					1.00	0.667	1.00	0.667	2.00	0.5
26........		5.14	7.06	26.60	7.06						
27........	3					1.00	0.667	1.00	0.667	2.00	0.5
28........		5.14	7.06	26.60	7.06						
29........	2										

This frame, therefore, is typical of what can be expected in reinforced concrete buildings designed for maximum economy, and meeting the necessary requirements as to space and clearances. The members differ widely in their relative stiffness. In the lower stories the columns are much stiffer than the girders. All these factors operate to slow down the convergence of the moment distribution.

The details of the computations are too lengthy to be reproduced in full. Points *a, b,* and *c* (Fig. 14(*a*)) were selected as typical.

Method 1.—The first attack was made along the lines suggested by Clyde T. Morris,[21] M. Am. Soc. C. E.; that is, the wind moment in a story was apportioned among the columns of that story according to their relative stiffness. No moment was apportioned to the girders. Second, the moments around each joint were balanced, which transferred some of the column moment into the attached beams. Due to their relatively small stiffness ratios, only a small percentage was transferred in any one step, which slowed the convergence. As the third step, the corresponding carry-over quantities were added. This completed one cycle. After all the stories were similarly balanced, a second cycle was started in the same way. Curves 1 to 3, Fig. 14, indicate the rate of convergence for successive cycles by this method. It should be remarked that because of the haunches on the girders, the carry-over factors differ from the customary 0.5 of the prismatic section (see Table

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[21] *Transactions,* Am. Soc. C. E., Vol. 96 (1932) p. 66.

6). It was found that after twenty-three cycles of operations, the results were still a considerable distance from a final value. After each cycle of operations the resultant moment is plotted against the number of the cycle. The values approach the true values at a decreasing rate, and any estimate as to the probable number of cycles required will fall short unless one can foresee how rapidly the rate of convergence is decreasing.

As the work progressed, attempts were made to forecast the final, probable values, with the idea of eliminating the intermediate steps. Professor Grinter's suggestion of proportioning from the rule:

$$\text{Final moment} = \text{Balanced moment} \times \frac{\text{True shear}}{\text{Calculated shear}}$$

was adopted after each cycle. In Curves 4 and 5 these estimates of the probable final values are recorded as "adjusted." It is to be noted that after a few cycles the estimates are amply close for practical purposes, although rather short of true mathematical agreement.

It would appear, judging from this single, asymmetrical instance, that satisfactory results can be obtained by the application of Professor Grinter's proportion even before the moment distribution has become steadied sufficiently to indicate the final values; but about twelve to fourteen cycles were necessary to make these adjusted moments reasonably close.

Method 2.—The foregoing method is too tedious to permit its application to actual design problems. The mere number of computations, even if they are of the simplest arithmetic, leaves a feeling of doubt. Fortunately, errors tend to work themselves out under repeated distributions; but the method is unduly cumbersome.

The slowness is caused by the taking of only a small amount of the column moment at each joint into the beams at each cycle. If the distribution percentage were larger, the convergence would be more rapid. By beginning with a fairly large value in the girders, considerable preliminary work would be eliminated. Arbitrary values could be assumed, but the results might then tend to converge on these values without their being the true results.

It was decided to start with the values obtained by the ordinary approximate solution. These values were first adjusted because it was impossible to have the same moments in the haunched beams and in the full depth spandrel. Consequently, about 10% was deducted from the approximate moments in each of the haunched spandrels and the corresponding amount was added into the full depth spandrel. This unbalanced the joints, and a rough adjustment of the column moments was made to keep each joint approximately in balance. In some cases, this was impossible. From a common sense viewpoint it was unreasonable to have a greater moment at the top of a column than at the bottom, and yet no other solution would balance the joint. In such cases values were adopted that were as nearly in balance as possible, and, at the same time, as nearly as possible consistent with each other. No careful attempt was made to fill either criterion, as it was felt that a few cycles would adjust the discrepancies.

With these trial values as a start, the same cycle of operations as in Method 1 was gone through seven times. With the girder moments assumed at something like their true values, the convergence is much more rapid. In Curves 6, 7, and 8, are plotted the complete set of computations through seven cycles for the same points, *a*, *b*, and *c*. Here, again, the reproduction of all the detail figures is of no help in visualizing the results. These curves offer a comparison with the previous method.

The approximations of the true final values by the proportion of true shear to calculated shear was applied to the moments in this case, and plotted as Curves 9, 10, and 11, in Fig. 14.

As the values were assumed approximately in balance around each joint and with the columns taking approximately the story moment, the convergence was quick—but not, apparently, on the best, true values. There was a marked tendency to close on the original assumptions. While the adjusted results appear fairly good, the method is mathematically unsound, and the apparent value is based only on a rather fortunate guess in setting up the original assumptions. Actually, the angles between beams and columns are not maintained as true right angles in this method; some relative angle change takes place.

Method 3.—This method followed exactly the author's suggestions for very irregular bents; that is, the *r* and *n*-values for each floor were computed and the total story moment was distributed as suggested. The *K*-values in Table 6 are relative only, and are about one-forty-seventh of the true $\frac{I}{L}$ -values. This does not affect the figures, but simply reduces all values in this ratio. For this reason, it is not possible to plot the various cycles. The results for the three points, *a*, *b*, and *c*, are recorded in Table 7, and the speed with which the figures converge is readily apparent. Six cycles of operations completed the distribution at each point to a precision of about 1 to 2 per cent. The final results at Points *a*, *b*, and *c* are computed by multiplying the values obtained as described, by the ratio of the true shear to the calculated shear; that is, by 7.37 for the fourth-story columns, 8.50 for the fifth-story columns, and the average for the fifth-floor beams.

TABLE 7.—VALUES OF DISTRIBUTED MOMENT AT POINTS *a*, *b*, AND *c* IN FIG. 14(*a*)

Cycle	*A*		*B*		*C*	
	Distributed	Result	Distributed	Result	Distributed	Result
	....	+116.0	......	0	....	+436.0
1.......	−78.5 −39.3	.... − 1.8	−12.2 − 7.4	...... −19.6	−310.0 −155.0	.... − 29.0
2.......	+16.8 + 8.4	.... + 23.4	+ 2.4 + 0.8	...... −16.4	+ 37.3 + 18.7	.... + 27.0
3.......	− 2.0 − 1.0	.... + 20.4	− 0.3 + 0.1	...... −16.6	+ 2.5 + 1.3	.... + 30.8
4.......	− 0.1 − 0.1	.... + 20.2	0.0 − 0.1	...... −16.7	− 3.3 − 1.7	.... + 25.8
5.......	+ 0.2 + 0.1	.... + 20.5	0.0 0.0	...... −16.7	+ 1.2 + 0.6	.... + 27.6
6.......	− 0.1 − 0.05	.... + 20.35	0.0 0.0	...... −16.7	− 0.3 − 0.2	.... + 27.1

The resultant moments were computed, as follows:

$$M_a = -20.35 \times 7.37 = 149.9$$

$$M_b = -16.70 \times 7.93 = 132.5$$

and,

$$M_c = +27.10 \times 7.37 = 199.7$$

At the fifth-floor joints the average value of $E\ \theta$ was equal to 4 960. Table 7 begins with no wind moment in the beams and with column moments obtained from approximate lateral movement as suggested by the author. These final results, only, are plotted as Curves 12, 13, and 14, in Fig. 14.

Method 4.—To check the other methods an analysis was made by slope deflections. The labor involved makes this method prohibitive for the actual design of structures. It is useful, however, as a criterion to measure the degree of approximation of the other methods.

In this case, the solution of the resulting simultaneous equations required considerable care, as the wide differences in the relative stiffnesses caused considerable variation in the coefficients, requiring nine significant figures for a satisfactory degree of precision.

The results at Points *a*, *b*, and *c*, are plotted as constants across all the cycles of Fig. 14 (see Curves 15, 16, and 17.) For comparison with Method 3, the average value of $E\ \theta$ for the fifth-floor joints was 4 920.

Conclusions.—For practical purposes with a bent of three spans of this type, the ordinary assumptions of Fleming's methods give reasonably satisfactory results. They afford the only method for making a preliminary design to use as a basis for more nearly exact elastic computations.

The method of starting with no wind moment in the girders and gradually transferring from the columns into the girders will bring convergence fairly rapidly in simple, symmetrical bents in which the relative stiffnesses of the different members do not vary too much.

When the members vary widely in their relative stiffnesses, the method of starting with no moment in the girders is too slow and tedious a process. In this particular case, twenty-three cycles of operations failed to come very close to the desired results.

By carrying through a few cycles and then proportioning the final values on the basis of the calculated moment in the columns to the true story moment, a number of cycles is saved. If the proportioning is done three or four times, using successive moment values after each cycle, a trend is established which indicates the final value more accurately.

More rapid convergence is obtained by starting with moments as nearly as possible like the final values in the girders. The convergence is rapid, but appears mathematically unsound because there is a marked tendency to converge on the assumed values. If these happen to be nearly correct (and with a little care in selecting them they should be), the results look good, but actually the method does not maintain the right angles between the beams and columns unchanged.

The author's method of approximating the actual story deflections from the relative stiffnesses of the beams and columns combined, and using this deflection to approximate the column moments, gives a much more rapid convergence than the original method of moment distribution. The results seem to be satisfactory. The author stated that some variation in relative stiffness was permissible. In this particular case, the stiffest column is more than 2 700% of the spandrel beam—a wide variation, and yet not more than can be anticipated in concrete structures.

Too many practical considerations are involved to make a high degree of precision necessary or desirable. A method that gives the necessary data for selecting members, rather quickly and sufficiently close for practical purposes, is what is desired.

It appears that one of Fleming's methods is quite close. If a check or more precise determinations is desired, the author's suggested method is by far the simplest. Although the tabulated computations appear rather simple, it should be remembered that they represent only three points, and the detailed computations involve several thousand sets. Approximately 75% of the time of Method 1 was saved in Method 4.

While it is unsound to judge from one or two isolated cases, the results as obtained from this one study are very favorable.

A. Floris,[22] Esq. (by letter).—In this interesting paper the author analyzes the wind stresses in the frames of tall buildings by the aid of the Cross method of balancing fixed-end moments. Furthermore, he states that an exact analysis of these frames requiring the solution of simultaneous equations — as, for instance, the slope deflection method used by Wilson and Maney — is a rather tedious process. For this reason he proposes the application of a less accurate method of moment distribution to the analysis of multiple-story frames subjected to wind pressure.

The present analysis, based as it is on the repeated distribution of the end moments, is to some extent a trial-and-error method. Such indirect methods, however, require considerable time, and their application to practical problems is rather discouraging. Their use can only be justified if direct and speedy methods of analysis are not available.

In the present case this is not necessary, because a direct method[23] has been developed by Professor F. Takabeya which excels in brevity and accuracy all other methods known to the writer. The necessary elastic equations derived by the slope deflection method can be written almost automatically and solved by the principle of iteration. This method of solving simultaneous equations permits the use of the slide-rule and is convenient and simple.

The author's statement regarding the difficulties arising in the analysis of multiple-story frames subjected to an additional translation of the joints, is not quite correct. Such frames are analyzed quite easily by the use of the

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[22] Civ. Engr., Los Angeles, Calif.

[23] "Methode der Gleichungstabellen zur Berechnung der Rechteckrahman," von Fukehei Takabeya, *Proceedings*, World Eng. Congress, Tokyo, 1929, Vol. VII, Pt. I, p. 277; also, "Rahmentafeln," von Fukehei Takabeya, Berlin, 1930, and "Das Verfahren der Gleichungstabulierung zur Berechnung des durchlaufenden Rahmens mit verschieden hohen senkrechten Stielen," von F. Takabeya, *Der Bauingenieur*, 1933, pp. 92 and 126.

Takabeya method, for instance, with little additional labor, and the calculations are not difficult. Furthermore, the claim that the method of balancing fixed-end moments only involves simple arithmetic is not quite true. In the last analysis any method arranged for practical use requires the application of simple arithmetical processes. For this reason the writer believes that, as an important practical advantage of the method under consideration, this fact is over-emphasized by the author.

Robins Fleming,[24] Esq. (by letter).—Professor Grinter presents a model arrangement of data and calculations. By making a table of story heights and K-values (Table 1) and writing on the diagram of the bent (Fig. 4) at each joint in a rectangle drawn around that joint the percentage factor for adjusting moments, confusion is avoided by the diagram not being cluttered with figures.

To assume that the center girders of a symmetrical bent are hinged at mid-point is an ingenious way of avoiding carry-over moments in these girders.

Professor Grinter's purpose is to present a simplified method of wind analysis. It is not difficult of application, but the designer should understand its limitations. Professor Grinter finds in the lower four stories of the classic Wilson and Maney bent the "criterion ratios" to correspond within a ratio of 11% for adjacent stories. The final or "true" moments are thus found to be within 7% of those obtained by the slope deflection method.

Variations of criterion ratios may be quite different in other cases. M. S. Ketchum, M. Am. Soc. C. E., has offered[25] a problem in which the bending moments of a three-aisle, six-story building are determined by a method outlined by Clyde T. Morris, M. Am. Soc. C. E. Using the K-values given and following the method of Professor Grinter, the criterion ratios vary from 5 to 50 per cent. The ratio in a story is sometimes less and sometimes greater than in the story above. Professor Grinter allows a variation of 15 per cent.

For more than twenty years the writer has sponsored, although he did not originate, the two conventional methods of obtaining wind stresses known as the cantilever and the portal methods. At the same time, he has always insisted that super-tall buildings and those of irregular framing should receive special study. Professor Grinter and other writers call attention to "errors" of 30% and even 100%, in the stresses obtained by the conventional methods. What they mean to say is that they differ by these percentages from those obtained by the slope-deflection method. The correctness of the slope-deflection method is vigorously disputed[26] by J. A. Van den Broek, M. Am. Soc. C. E. David A. Molitor, M. Am. Soc. C. E., who is often quoted, does not regard the basic assumptions of the method as "exact."[27]

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[24] Care, American Bridge Co., New York, N. Y.

[25] "Steel Mill Buildings", by M. S. Ketchum, Fifth Edition, 1932, Chapter on "Stresses in Stiff Frames by Moment Distribution".

[26] "Elastic Energy Theory", John Wiley & Sons, Inc., 1931.

[27] *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 189.

Notwithstanding an occasional assault the consensus of engineering opinion is that it is the most nearly correct of all methods thus far presented. It is not workable, however, for design purposes. The Cross methods, although tedious, are feasible and are valuable for analysis of stresses obtained by other methods. For the purpose of analysis, Professor Grinter's paper is especially valuable.

J. D. GEDO,[28] ESQ. (by letter).—Under the heading, "The Open-Web or Vierendeel Truss," the author states that fixed-end moments in this type of truss occur only in the horizontal members or chords, "provided the effect of direct stress deformation is to be neglected, as is usual."

To neglect the effect of direct stresses is permissible only if they are small in comparison with the moments. No one would neglect the direct stresses in the case of arches; nor is it permissible to do so in the case of Vierendeel trusses.

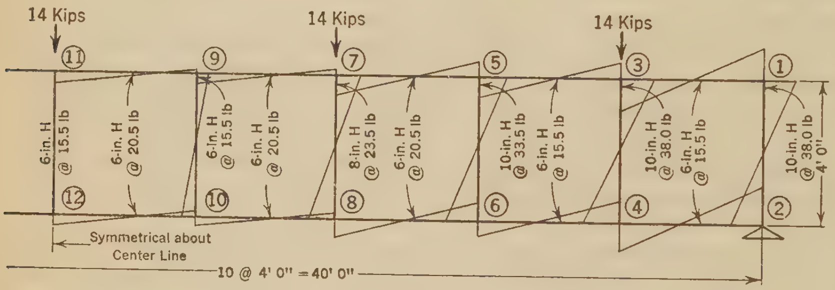

FIG. 15.

The writer solved the example in Fig. 15 by the theorem of least work both by neglecting and considering the effect of direct stresses (in the chords). In Table 8, which gives the results of this study, the subscripts, *A*, *B*, *L*, and *R*, signify "above," "below," "left," and "right," respectively; for instance, M_{8B} is the moment immediately below Joint 8.

TABLE 8.—COMPARISON OF MOMENTS IN VIERENDEEL TRUSSES

(1 Kip = 1 000 Pounds)

Moment, *M*, at point: (See Fig. 15)	MOMENT, IN KIP-FEET; DIRECT STRESS IN CHORDS:		Moment, *M*, at point: (See Fig. 15)	MOMENT, IN KIP-FEET; DIRECT STRESS IN CHORDS:		Moment, *M*, at point: (See Fig. 15)	MOMENT, IN KIP-FEET; DIRECT STRESS IN CHORDS:	
	Neglected	Considered		Neglected	Considered		Neglected	Considered
1........	35.51	35.10	$5_L = 6_L$....	21.75	20.18	$9_B = 10_A$..	11.26	11.47
2........	35.51	35.10	$5_R = 6_R$....	21.15	22.17	$9_L = 10_L$..	4.37	2.73
$3_B = 4_A$...	55.34	54.73	$7_B = 8_A$....	27.36	27.08	$9_R = 10_R$..	6.89	8.74
$3_L = 4_L$...	20.85	19.83	$7_L = 8_L$....	7.11	5.26	$11_B = 12_A$..	0.00	0.00
$3_R = 4_R$...	34.49	34.90	$7_R = 8_R$....	20.25	21.82	$11_R = 12_R$..	9.63	11.27
$5_B = 6_A$...	42.90	42.35						

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[28] Structural Designer, Care, Alexander D. Crosett, Cons. Engr., New York, N. Y.

It is to be noted, in comparing moments in which direct stresses are considered and neglected, that errors as great as 37% (see M_{9L}) can occur.

H. V. Spurr,[29] M. Am. Soc. C. E. (by letter).—The writer agrees with the author (see "Synopsis") that:

> "One of the most difficult problems that the structural engineer must solve is the analysis of stresses in a continuous frame in which joint displacements occur. * * * The calculation of wind moments in tall building frames is such a problem."

He further agrees (see "Use of the Simplified Method of Wind Stress Analysis") that "the experienced designer has a real advantage over the beginner in the use of this method;" or, for that matter, in the use of any method. Furthermore, Professor Grinter is very wise when he states that "the design of any indeterminate structure should be simplified by a previous knowledge of correct proportions." The writer considers it unfortunate that the author did not base his entire paper on these essential facts, since it can be said with equal truth that the anaylsis of an existing structure will be simplified by a previous knowledge of correct proportions.

It can rightly be said that the Cross method has its greatest value as a means of acquiring a visualization of the action of continuous frames, which will assist in developing and perfecting this so-called "previous knowledge" of the engineer, which is so essential to successful design or analysis. Once acquired, this knowledge is so potent as to render the use of the method in detail often both tedious and superfluous. For this reason, the writer must take serious exception to the statement under the heading, "Methods in Common Use," that "to date, the most usable method of wind stress analysis, which also can claim to be more than a rough estimate, is that suggested by Ross and Morris."

It can safely be said that practical design methods are in use in more than one office, that will produce results, the general accuracy of which can not be seriously questioned from the standpoint of analysis. These methods are based on sound mechanics and are used with a visualization of the elastic behavior of continuous frames. Their merit lies in the freedom from arbitrary and incorrect assumptions commonly used in connection with mathematical methods as such. Much confusion and error have occurred in the past from careless assumptions used to make the mathematics apply to a given case.

In the light of "a previous knowledge of correct proportions" how would an experienced engineer initiate his analysis of an existing high building frame? The procedure might be, as follows:

(*a*) Allocate a wind load to each bent that will be tied in later with the design load on the structure.

(*b*) Calculate the horizontal wind shears in the various bents in each story.

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[29] Chf. Engr., Purdy & Henderson Co., New York, N. Y.

(*c*) Distribute the horizontal shears into the various panels on the basis of beam distortion only, assuming points of inflection at the mid-span points of the beams. Such shears will cause end moments in the beams, which, in turn, will produce extreme fiber stresses in bending inversely proportioned to the span-depth ratios of the beams (based on their clear spans between columns).

(*d*) From the beam moments the column moments and shears may be obtained, on the basis of points of inflection occurring at mid-story points in the columns.

(*e*) The participation of the various columns in the web distortion may be obtained on the basis of these moments and shears. Their relative values may be obtained, and their relation to beam participation in web distortion as well. If the columns participate equally, or if the column participation is small relative to the beam participation, one may proceed to the next step without review.

(*f*) Determine the relative rigidity of the beams from floor to floor based on the assumed moments. If the floors have approximately the same relative rigidity under their respective moments from wind assumed, then the analysis so far is approximately correct, except for the influence of connections, which is a detail not mentioned by the author. A study of the values of relative rigidities between beams and columns and between floors will reveal how the results are to be modified in specific cases. Frequently, no modification is necessary.

Before any such modification is made, however, it is of vital importance to ascertain the possible influence of change in the length of the columns on the entire analysis. If it is found from the analysis at this stage that the assumed shear distribution will induce vertical stresses in the columns, which, in turn, would cause a vertical movement of the joints in such a manner as to throw the various floors more or less out of line, it is useless to make any further refinement in the analysis without taking such action into account.

How this is to be accomplished the author has not seen fit to establish. In many cases, it will be a weary and tedious task if based on conventional methods of correcting and recorrecting for joint displacement in a high irregular frame. For this reason, the writer has paralleled the procedure outlined by the author with one of his own, up to the point of considering column axial deformation. It is hoped that the points of attack outlined may have some useful significance.

In his discussion[30] of the Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind Bracing, the writer has pointed out the significance of correct proportions in the design and analysis of multi-story frames, and has indicated a simplified use of the Cross method of analysis under certain circumstances. What needs to be emphasized is that the six steps in the analysis outlined in the present discussion will disclose the type of structure with which one has to deal. The degree of its irregularity will be revealed immediately.

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[30] *Proceedings*, Am. Soc. C. E., August, 1932, p. 1126.

If the structure is regular and well-proportioned the results obtained in the first analysis will be substantially correct, and will be reached in a fraction of the time required by either the general method of successive corrections, or by the author's simplified method. Furthermore, it is important to know as quickly as possible in what manner and do what degree the structure is irregular or unusual. In fact, it is absolutely essential for the engineer to sense these facts, in order to use the simplified method at all, since it is based on: (1) The calculation of the true deflections under the applied loads; (2) producing these deflections by bending in the columns only, with joints fixed against rotation; and (3) balancing the moments in succession at all joints.

It is perfectly true that Steps (2) and (3) are simple of execution, but it is these two steps only which involve the Cross method at all. The real difficulty comes in Step (1) which actually involves the approximate solution of the problem. It is true that under Step (1) the solution obtained may, through lack of insight, be somewhat inaccurate, in which case Steps (2) and (3) will reveal the direction toward which the analysis must move to improve the accuracy. The time consumed in the analysis, however, will depend largely upon the efficiency shown in Step (1), which is all-important.

For this reason the writer believes that progress in the analysis of irregular frames is frequently most rapid through a careful study of deflections or drift produced in the web system by the applied loads, through an intelligent spotting of the positions of the points of contraflexure, and then checking the solution for the condition of minimum deflection in the structure as a whole. This is a common-sense use of the method of least work bearing in mind that the structure will perform under the applied loads in the most rigid manner possible. Frequently, a given structure may have a marked irregularity, but in one respect only, and a reasonable solution is quickly obtained by inspection, or by bracketing the true solution by two analyses somewhat in the same manner that artillery fire is directed.

The author has made a valuable contribution to the art of analysis, but has under-emphasized the degree of art involved in the analysis or design of continuous structures of irregular character. This common error in emphasis is characteristic of nearly all technical papers and discussions on the subject; and it springs from a natural desire of authors and engineers to substitute a definite mathematical procedure for a trained reflection and visualization of the conditions affecting the correct solution.

It should be realized that the actual geometry of deflection is influenced by the clear spans of the members, by the character of the connections, whether angles, split beams, or knee-braces, and, in high towers, that it is largely controlled by the elastic action of the columns under axial stresses. Neither the Cross method nor the slope deflection method are, in themselves alone, suitable instrumentalities for many problems encountered in practice.

JOHANNES SKYTTE,[31] M. AM. SOC. C. E. (by letter).—This paper is of unquestionable value for the analysis of framed structures and particularly for tall buildings acted upon by lateral forces.

In connection with the application of the simplified method for irregular structures, it is of importance to estimate deflections and, for this reason, the following approximate formulas will be introduced: (I) Formula for determining location of point of counterflexure in columns; (II) formula for determining location of point of counterflexure in girders; and (III) formula for determining ratio of girder shears.

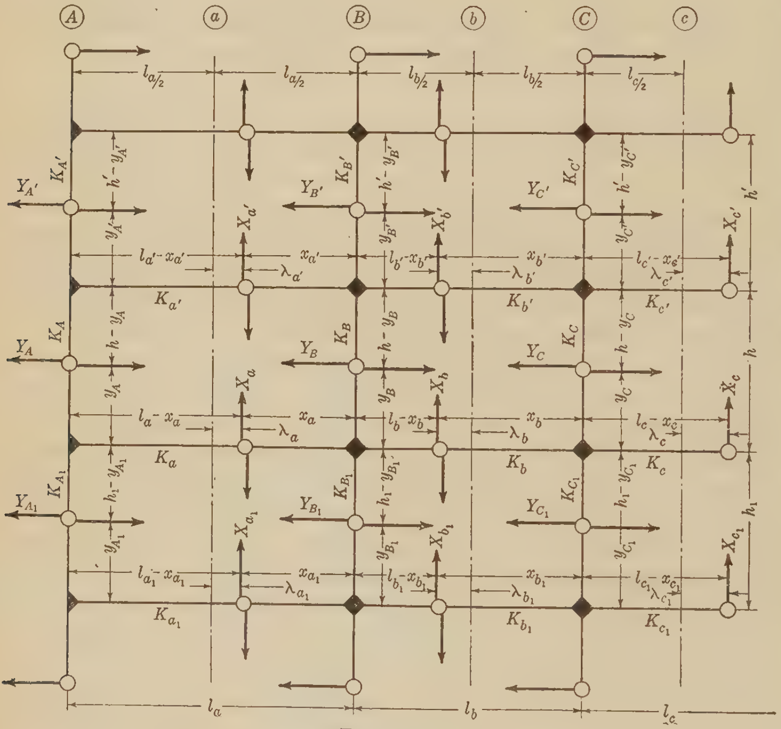
A
a
B
b
C
c
$l_{a/2}$
$l_{a/2}$
$l_{b/2}$
$l_{b/2}$
$l_{c/2}$
$K_{A'}$
$h'-y_{A'}$
$Y_{A'}$
$y_{A'}$
$K_{B'}$
$h'-y_{B'}$
$Y_{B'}$
$y_{B'}$
$K_{C'}$
$h'-y_{C'}$
$Y_{C'}$
$y_{C'}$
$X_{a'}$
$X_{b'}$
$X_{c'}$
h'
$l_{a'}-x_{a'}$
$x_{a'}$
$\lambda_{a'}$
$l_{b'}-x_{b'}$
$x_{b'}$
$\lambda_{b'}$
$l_{c'}-x_{c'}$
$\lambda_{c'}$
$K_{a'}$
$K_{b'}$
$K_{c'}$
K_A
$h-y_A$
Y_A
y_A
K_B
$h-y_B$
Y_B
y_B
K_C
$h-y_C$
Y_C
y_C
X_a
X_b
X_c
h
l_a-x_a
x_a
λ_a
l_b-x_b
x_b
λ_b
l_c-x_c
λ_c
K_a
K_b
K_c
K_{A_1}
$h_1-y_{A_1}$
Y_{A_1}
y_{A_1}
K_{B_1}
$h_1-y_{B_1}$
Y_{B_1}
y_{B_1}
K_{C_1}
$h_1-y_{C_1}$
Y_{C_1}
y_{C_1}
X_{a_1}
X_{b_1}
X_{c_1}
h_1
$l_{a_1}-x_{a_1}$
x_{a_1}
λ_{a_1}
$l_{b_1}-x_{b_1}$
x_{b_1}
λ_{b_1}
$l_{c_1}-x_{c_1}$
λ_{c_1}
K_{a_1}
K_{b_1}
K_{c_1}
l_a
l_b
l_c

FIG. 16.

(I) Formula for Determining Location of Point of Counterflexure in Columns.—For Column *A*, Fig. 16:

$$\frac{M_{\text{top}}}{M_{\text{bottom}}} = \frac{h - y_A}{y_A} = \frac{3h + \alpha\left(\frac{h_1}{2} + h\right) - \beta\,\frac{h'}{2}}{3h + \beta\left(\frac{h'}{2} + h\right) - \alpha\,\frac{h_1}{2}} \qquad \text{.........(7)}$$

in which, $\alpha = \frac{K_A}{K_a}$ and $\beta = \frac{K_A}{K_{a'}}$. Equation (7) will also apply to Column *B*,

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[31] Hydr. Eng. Designer, City of San Francisco, Hetch Hetchy Dept., San Francisco, Calif.

Fig. 16, when $\alpha = \frac{K_B}{K_a + K_b}$ and $\beta = \frac{K_B}{K_{a'} + K_{b'}}$. Similar results follow for any other interior column.

If a single combined column, restrained by a single girder at each floor level, is considered, then:

$$\frac{M_{\text{top}}}{M_{\text{bottom}}} = \frac{3h + \alpha\left(\frac{h_1}{2} + h\right) - \beta\frac{h'}{2}}{3h + \beta\left(\frac{h'}{2} + h\right) - \alpha\frac{h_1}{2}} \quad \ldots\ldots\ldots\ldots\ldots (8)$$

in which,

$$\alpha = \frac{(K_A + K_B + K_C + \ldots)}{2\,(K_a + K_b + K_c + \ldots)}\text{; and } \beta = \frac{(K_A + K_B + K_C + \ldots)}{2\,(K_{a'} + K_{b'} + K_{c'} + \ldots)}$$

Equation (8) will give closer results than the formula preceding Fig. 7 of the author's paper. It is to be noted that $\alpha = 0$ in the basement story if the columns are fixed at the bottom. Applying Equation (8) to the American Insurance Union Building the results given in Table 9 are obtained (in thousands of foot-pounds).

TABLE 9.—COLUMN MOMENTS BY EQUATION (8)

Story	TOTAL COLUMN MOMENT		Story	TOTAL COLUMN MOMENT	
	Top	Bottom		Top	Bottom
6	1 550	1 550	2	3 030	2 040
5	1 700	1 680	*M*	2 300	2 130
4	1 680	1 650	1	2 810	3 750
3	1 590	2 230	*B*	3 750	4 120

(II) Formula for Determining Location of Point of Counterflexure in Girders.—In Bay *a*, Fig. 16,

$$\frac{l_a - x_a}{x_a} = q\,\frac{1 + r}{3 - r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which, $r = \frac{5 + q + 2\,\beta}{2\,(q^2 + q + 1) + \alpha\,q^2 + \beta}$; $q = \frac{y_A + (h_1 - y_{A1})}{y_B + (h_1 - y_{B1})}$; $\alpha = \frac{2\,K_a}{K_A + K_{A1}}$; and $\beta = \frac{2\,(K_a + K_b)}{K_B + K_{B1}}$.

Equation (9) will also apply for Bay *b*, Fig. 16, when $q = \frac{y_B + (h_1 - y_{B1})}{y_C + (h_1 - y_{C1})}$; $\alpha = \frac{2\,(K_a + K_b)}{K_B + K_{B1}}$; and $\beta = \frac{2\,(K_b + K_c)}{K_C + K_{C1}}$. Similar results follow for any other interior bay.

(III) Formula for Determining Ratio of Girder Shears.—Let *b* and *c* be two adjacent bays in a bent; then:

$$\frac{X_b}{X_c} = \frac{K_b}{K_c}\,\frac{l_c \pm 6\,\lambda_c}{l_b \pm 6\,\lambda_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

In order to determine the sign of λ, it is necessary to consider oneself located at Column C, Fig. 16; then the plus sign applies when the column and the point of counterflexure in the adjoining girders are on opposite sides of the center line of the same girders. The minus sign applies when they are on the same side of the center line. For the sake of illustration apply this rule to the ratio of girder shears in Fig. 16, then: $\dfrac{X_a}{X_b} = \dfrac{K_a}{K_b}\,\dfrac{l_b - 6\,\lambda_b}{l_a - 6\,\lambda_a}$ and $\dfrac{X_b}{X_c} = \dfrac{l_c + 6\,\lambda_c}{l_b + 6\,\lambda_b}$.

JOHN B. WILBUR,[32] ASSOC. M. AM. SOC. C. E. (by letter).—Step (4) of the simplified method of wind stress analysis presented by Professor Grinter, consists of computing the total shear in each story from the balanced moments of Step (3). This is easily done by dividing the sum of the end moments of the columns of a story by the story height, providing one assumes that the wind loads are concentrated at the panel points of a bent. In some cases, however, it might be more correct to assume that the wind load is distributed over the windward column stack of the bent, or partly distributed in this way and partly applied in panel loads.

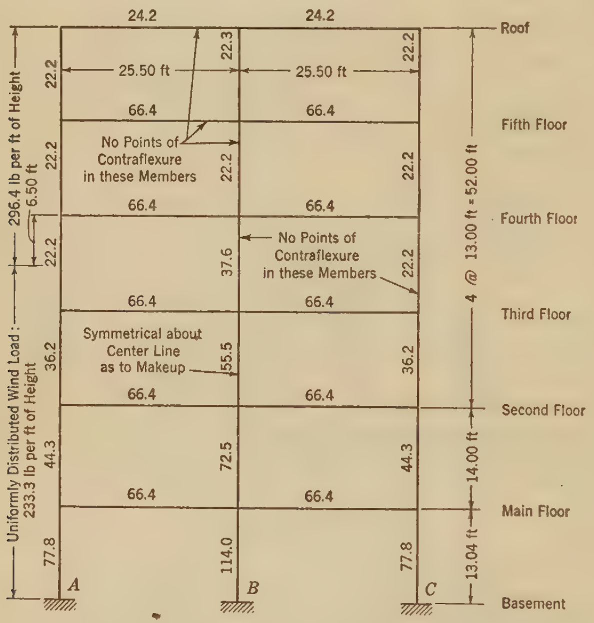
24.2
24.2
Roof
22.3
22.2
22.2
25.50 ft
25.50 ft
66.4
66.4
Fifth Floor
No Points of Contraflexure in these Members
22.2
22.2
22.2
66.4
66.4
Fourth Floor
6.50 ft
22.2
37.6
No Points of Contraflexure in these Members
22.2
66.4
66.4
Third Floor
Symmetrical about Center Line as to Makeup
36.2
55.5
36.2
66.4
66.4
Second Floor
44.3
72.5
44.3
66.4
66.4
Main Floor
77.8
114.0
77.8
A
B
C
Basement
Uniformly Distributed Wind Load: 296.4 lb per ft of Height
233.3 lb per ft of Height
4 @ 13.00 ft = 52.00 ft
14.00 ft
13.04 ft

FIG. 17.—DIMENSIONS, AND STIFFNESS FACTOR, K, OF A BUILDING FRAME (AVERAGE VALUES OF K ARE GIVEN FOR GIRDERS).

That the assumption made with regard to this condition may play an important rôle in determining the computed wind stresses in a bent is illustrated by the following: Consider the moments in the columns and girders of the bent shown in Fig. 17 due to the given distributed wind loads. The moments in Table 10(*a*) were determined by means of a model and deformation apparatus.[33] These results are unusual in two respects: There is no tendency toward symmetry of stress in a symmetrical bent, and several of the members have no points of contraflexure. These conditions can be explained partly by the fact that the model test takes into account (although incorrectly) deflections due to direct stress and shear. The writer did not believe that the effect of these items was sufficiently great to offer complete explanation for the unusual results. A solution was made, therefore, by slope deflection, concentrating the same total wind loads at the panel points, and, to simplify the tedious solution, using average moments of inertia for girders with cover-plates. The results of this solution, which was carried out on a computing machine with considerable accuracy, are given in Table 10(*b*), and show such completely different moment values, that the importance of the manner of loading assumed is apparent at once.

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[32] Instructor in Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.

TABLE 10.—EFFECT OF VARIATIONS IN THE MANNER OF DISTRIBUTING WIND LOADS

Floor	Wind load	MOMENTS, IN FOOT-POUNDS, ACT CLOCKWISE AT END OF MEMBER IF SHOWN POSITIVE									
		COLUMNS (SEE FIG. 17)						GIRDERS (SEE FIG. 17)			
		A		*B*		*C*		*AB*		*CD*	
		Top	Bottom	Top	Bottom	Top	Bottom	Left	Right	Left	Right
(*a*) MOMENTS BY MODEL TEST AND DEFORMATION APPARATUS (DISTRIBUTED LOADS)											
Roof....								+4 140	—1 410	+1 200	+2 560
	296.4*	—4 140	—6 810	+200	—9 500	+2 560	—2 185				
5.....								+21 990	—3 100	+12 170	+13 085
	296.4*	—15 180	—8 210	0	—31 450	—10 900	—9 460				
4.....								+33 593	+1 877	+12 170	+22 530
	296.4* 222.3*	—25 382	—22 128	+17 160	—68 900	—12 890	—11 678				
3......								+48 668	+37 730	+72 700	+6 880
	222.3*	—26 540	—24 260	—41 230	—35 270	+4 800	—40 800				
2......								+38 810	+58 200	+34 300	+60 700
	222.3*	—14 550	—33 820	—59 350	—6 450	—33 700	—68 500				
Main....								+51 955	+45 000	+4 050	+92 500
	222.3*	—18 135	—45 450	—42 600	—67 900	—24 000	—39 010				
Basement								....	....	....	....
(*b*) MOMENTS BY THE SLOPE-DEFLECTION METHOD (EQUIVALENT CONCENTRATED LOADS)											
Roof....	1 910†							+3 600	+3 000	+3 000	+3 600
		—3 600	—2 800	—6 000	—5 900	—3 600	—2 800				
5.....	3 830†							+14 700	+10 800	+10 800	+14 700
		—11 900	—10 600	—15 600	—14 100	—11 900	—10 600				
4.....	3 720†							+26 500	+23 100	+23 100	+26 500
		—15 900	—14 800	—32 000	—29 600	—15 900	—14 800				
3.....	3 010†							+37 000	+34 800	+34 800	+37 000
		—22 200	—20 200	—40 000	—37 100	—22 200	—20 200				
2.....	3 010†							+47 900	+45 400	+45 400	+47 900
		—27 700	—27 300	—53 700	—53 000	—27 700	—27 300				
Main....	2 940†							+49 900	+47 300	+47 300	+49 900
		—22 600	—43 100	—41 600	—67 500	—22 600	—43 100				
Basement								....	....	....	....

* Units are in pounds per foot of height. † Units are in pounds.

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[33] "An Example of the Mechanical Solution of a Statically Indeterminate Structure," by H. W. Coultas and V. H. Lawton, *The Structural Engineer*, October, 1930, p. 344.

In a given building frame, the paths which the wind loads follow in their transfer from exterior walls to wind bents depend on the type of construction used and the ratio of column spacing to story height. For skeleton steel buildings with curtain walls, the assumption that the wind loads are concentrated at the panel points of the bents seems rational to the writer, particularly if the building is tall, or if the ratio of column spacing to story height is large. The possible importance of this assumption in its effects on the computed stresses, however, should not be overlooked.

L. E. GRINTER,[34] ASSOC. M. AM. SOC. C. E. (by letter).—The constructive nature of the criticism apparent in nearly all the discussions of this paper is appreciated. The writer wishes particularly to thank Messrs. Letherbury and Reese for their carefully prepared discussions showing the application of the simplified method to the analysis of practical building frames. He is highly mindful of the labor involved in the preparation of these valuable discussions.

Professor Witmer's discussion refers primarily to the determination of the moments caused by column deformation. He states correctly that this effect can be evaluated with the effect of horizontal translation, but it should be noted that this procedure is possible only when the fixed-end column moments are obtained from a preliminary estimate of deflections and approximate column stresses are known.

Mr. Timby requests the answers to several questions which the writer attempted to answer in the original paper. Of course, it is not true that the preliminary location of points of contraflexure at the mid-heights of the columns, or the temporary assumption that the story deflection is proportional to the story height, carries through the entire analysis and vitiates the final results. These assumptions merely form a starting point in the analysis. The simplified method is based on successive corrections, and the final moments can be calculated to agree with any predetermined standard of accuracy.

Perhaps it is unfortunate that the word, "ideal," was used to describe a bent theoretically proportioned so as to deflect linearly. However, no suggestion was made that such a frame was desirable. In fact, its only purpose was to help the reader bridge the gap from the single-story bent to the tall building frame. Mr. Timby seems to have misunderstood the purpose of this part of the paper and digresses into a comparison of the ideal frame with a mechanical model in which he was particularly interested.

The results of the analysis of this model by means of the Beggs deformeter by Messrs. Coultas and Lawton were used by both Mr. Timby and Mr. Wilbur in drawing certain rather important conclusions. This was particularly unfortunate since it can be shown that the Coultas and Lawton analysis of this model bent is radically in error.

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[34] Prof., Structural Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

TABLE 11.—Comparison of Column Moments for Coultas and Lawton Bent as Obtained by the Simplified Method, the Slope Deflection Method, and by Mechanical Analysis

Story	Coultas and Lawton solution (use of Beggs deformeter) (1)	Slope deflection analysis (story loads and average *K*-values) (2)	Simplified method (story loads and average *K*-values) (3)	Simplified method (distributed load and average *K*-values) (4)	Simplified method (story loads and varying cross-sections) (5)	Simplified method (distributed load and varying cross-sections) (6)
Column *A* (Windward Column)						
6	+ 4 140	+ 3 600	+ 3 500	+ 1 000	+ 3 700	+ 1 200
	+ 6 810	+ 2 800	+ 2 700	+ 7 400	+ 2 700	+ 7 300
5	+15 180	+11 900	+11 800	+ 7 500	+11 600	+ 7 300
	+ 8 210	+10 600	+10 600	+14 700	+10 000	+14 000
4	+25 382	+15 900	+16 100	+12 300	+16 000	+12 300
	+22 128	+14 800	+14 700	+18 400	+15 600	+19 400
3	+26 540	+22 200	+22 300	+19 000	+21 400	+18 200
	+24 260	+20 200	+20 200	+23 500	+20 000	+23 400
2	+14 550	+27 700	+27 700	+24 200	+27 600	+24 200
	+33 820	+27 300	+27 300	+30 900	+28 900	+32 700
1	+18 135	+22 600	+22 800	+19 500	+20 400	+17 100
	+45 450	+43 100	+43 200	+45 500	+44 500	+47 000
Column *B* (Center Column)						
6	− 200	+ 6 000	+ 6 200	+ 5 300	+ 6 200	+ 5 400
	+ 9 500	+ 5 900	+ 6 200	+ 5 500	+ 5 600	+ 5 000
5	0	+15 600	+15 700	+15 900	+16 100	+16 200
	+31 450	+14 100	+13 900	+13 900	+15 400	+15 600
4	−17 160	+32 000	+32 200	+32 200	+30 000	+30 200
	+68 900	+29 600	+29 500	+29 600	+29 600	+29 800
3	+41 230	+40 000	+39 700	+39 800	+40 000	+39 900
	+35 270	+37 100	+37 000	+37 000	+39 200	+39 100
2	+59 350	+53 700	+53 800	+53 800	+50 800	+51 100
	+ 6 450	+53 000	+53 000	+53 000	+51 800	+52 200
1	+42 600	+41 600	+41 200	+41 200	+40 000	+40 200
	+67 900	+67 500	+67 200	+67 200	+70 200	+70 700
Column *C* (Leeward Column)						
6	− 2 560	+ 3 600	+ 3 500	+ 3 300	+ 3 700	+ 3 400
	+ 2 185	+ 2 800	+ 2 700	+ 2 400	+ 2 700	+ 2 400
5	+10 900	+11 900	+11 800	+11 900	+11 600	+11 700
	+ 9 460	+10 600	+10 600	+10 600	+10 000	+10 000
4	+12 890	+15 900	+16 100	+16 100	+16 000	+16 100
	+11 678	+14 800	+14 700	+14 700	+15 600	+15 700
3	− 4 800	+22 200	+22 300	+22 100	+21 400	+21 400
	+40 800	+20 200	+20 200	+20 300	+20 000	+20 000
2	+33 700	+27 700	+27 700	+27 700	+27 600	+27 800
	+68 500	+27 300	+27 300	+27 300	+28 900	+29 000
1	+24 000	+22 600	+22 800	+22 800	+20 400	+20 500
	+39 010	+43 100	+43 200	+43 200	+44 500	+44 800

The Coultas and Lawton bent was intended to represent a practical six-story building frame. Hence, the columns change section above the girder line, and the girders are reduced in cross-section near the columns. Thus, the analysis is complicated by intra-panel change in cross-section as well as by the fact that the wind force is taken as distributed over the windward column. An approximate analysis can readily be made by the simplified method by concentrating the loads at the floor levels and by using average *K*-values to represent the stiffness factors of the members.

The true carry-over factors and the distribution factors at the joints will be affected by intra-panel changes in cross-section. In order to obtain a more exact analysis than would result from average *K*-values, the actual factors should be used. Furthermore, the effect of the distributed loading in producing additional fixed-end moments in the windward column must be considered. Such analyses were performed, but are not included herein. It is sufficient to state that a comparison of column moments determined by four such analyses with those determined by Messrs. Coultas and Lawton, are given in Table 11 and show the latter to be so radically different from any of the others that one can only conclude that they are entirely valueless.

Since Mr. Timby's further observations and questions were based upon his study of this incorrect deformeter analysis, it seems unnecessary to take further space for their consideration. Mr. Wilbur observed that the deformeter analysis must take incorrectly into account the effect of direct stress and shearing deformation, but he was properly not convinced that this effect plus the effect of the distributed load could account for the unusual results shown in his Table 10. Table 11 shows the effect of load distribution and of intra-panel change in cross-section, and deflections measured on a model by Mr. M. P. Frank (formerly Instructor in the Civil Engineering Department of the Agricultural and Mechanical College of Texas, College Station, Tex.), prove that the effect of direct stress and shearing deformations are not serious for this particular bent. Lateral deflections computed by different methods were determined by Mr. Frank as shown in Fig. 18, in which Curve 5 represents the deflections computed from final moments obtained by the simplified method, using distributed loading and members with varying cross-sections. The deflection curve computed from the moments obtained by the slope-deflection method is identical with Curve 3, Fig. 18, of estimated deflections.

In regard to load distribution, the writer would like to emphasize the point that actual wind loading is not distributed entirely to the column. In fact, where the panel is 20 ft wide by 12 ft high, about two-thirds of the total wind pressure is concentrated at the floor levels by the spandrel beams, and, if the walls do not fit tightly against the columns, practically all the wind pressure may be thus concentrated. Hence, the writer feels that the moments of Column (5), Table 11, represent the most probable set of values. Mr. Wilbur seems to concur in this opinion.

Mr. Letherbury studied an interesting step-back frame by the simplified method and by the Bowman method. Despite the set-back, this is an extremely regular frame, which accounts for the small error (20%) shown by the statically determinate (Bowman) method. No such agreement could be expected where the variation of K-values corresponds to a normally irregular frame. Mr. Letherbury makes a valuable suggestion in regard to the use of the writer's method of estimating deflections for the study of tower deflections and vibrations.

Mr. Weiss emphasizes the time consumed by the wind-stress analysis of a tall building. He observes that the 56-story Irving Trust Company Building contains 6 200 wind-resisting members in its entire frame. The writer counts about 620 members in a single bent, as shown in Table 5, and, hence, he concludes that there must be about ten such bents. In a very regular building all ten such bents may be practically identical, but it will be assumed that there are ten variations to be analyzed separately. A wind-stress analysis of a tall frame can be performed by the simplified method in considerably less than 10 min per member. The time consumed would be about 500 hr if the frames were so irregular from top to bottom that a statically determinate analysis could not be used over any part of the frame. Is it unreasonable to detail five men for two or three weeks to obtain such information? Can not they be allowed two or three months, if necessary, in which to produce a frame truly proportioned to resist wind stresses in an economical manner? If the structural frame is irregular, is not the engineer justified in spending 0.1% of the cost of a skyscraper in really solving the wind-stress problem?

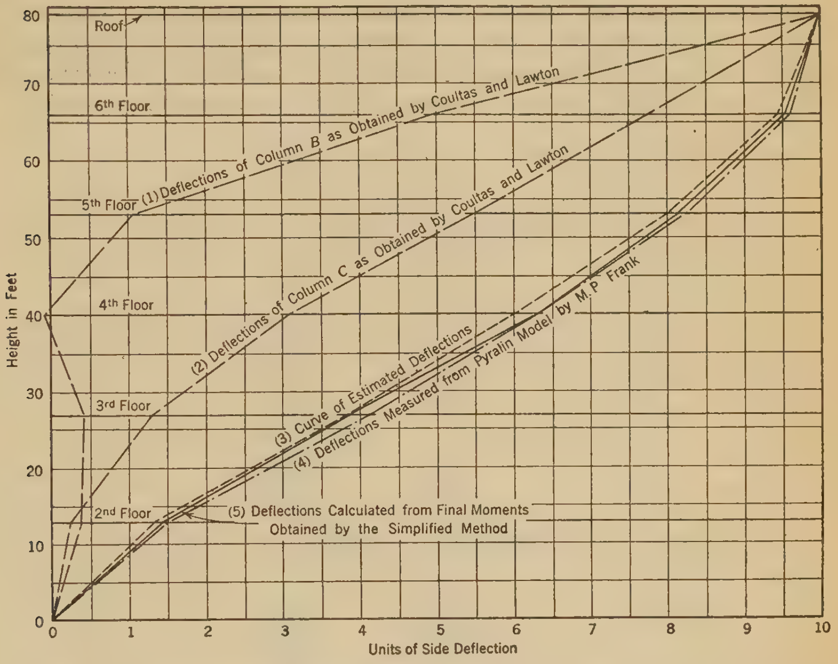

FIG. 18.—COMPARISON OF LATERAL DEFLECTIONS FOR COULTAS AND LAWTON BENT.

Mr. Weiss feels that the writer has "overlooked one of the peculiar characteristics of modern tall buildings." He refers to the use of stiff columns and slender girders in the lower stories. On the contrary, the simplified method was designed to apply to all such cases. Fig. 13 is purported to show that the task of applying the simplified method to a structure such as the Irving Trust Company Building would become an impossible undertaking because of slowness of convergence of moments. The writer believes that Mr. Weiss confused the simplified method and the general method of successive corrections, as adopted by Sub-Committee No. 31, Committee on Steel, of the Construction Division, on Wind-Bracing in Steel Buildings, because his criticism would be applicable to the latter, but not to the former. To show that the application of the simplified method to a frame with stiff columns and slender girders is not "such a Herculean undertaking as to overshadow completely the task performed by Professors Wilson and Maney in solving sixty simultaneous equations," the writer has analyzed the frame shown in Fig. 19. This frame maintains the same essential features of stiff columns and slender girders found in the Irving Trust Company Building. Deflections are estimated in Table 12 and moments are balanced in Fig. 20. An interesting characteristic is the development of negative moments at the top of the lower-story columns, which, incidentally, necessitated the introduction of correction moments into Story 1. However, the labor involved in the analysis is not particularly greater than would be necessary for a bent with relatively stiff girders. A time allowance of 10 min per member is ample.

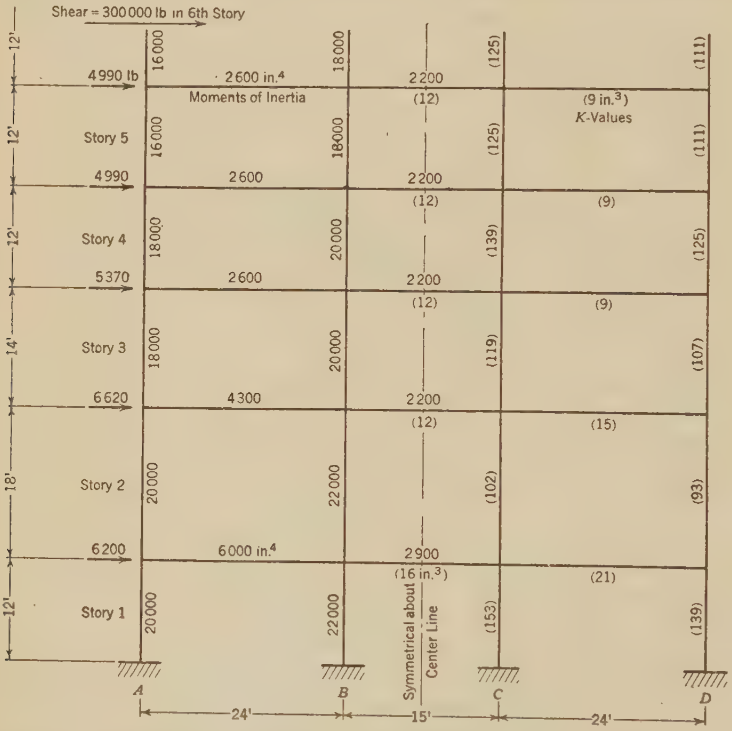
Shear = 300000 lb in 6th Story
12'
4990 lb
16000
2600 in.4
Moments of Inertia
18000
2200
(12)
(125)
(9 in.3)
K-Values
(111)
12'
Story 5
16000
18000
(125)
(111)
4990
2600
2200
(12)
(9)
12'
Story 4
18000
20000
(139)
(125)
5370
2600
2200
(12)
(9)
14'
Story 3
18000
20000
(119)
(107)
6620
4300
2200
(12)
(15)
18'
Story 2
20000
22000
(102)
(93)
6200
6000 in.4
2900
(16 in.3)
(21)
12'
Story 1
20000
22000
Symmetrical about Center Line
(153)
(139)
A
B
C
D
24'
15'
24'

FIG. 19.—PROPERTIES OF FRAME HAVING STIFF COLUMNS AND FLEXIBLE GIRDERS.

Mr. Reese has contributed a valuable study comparing the simplified method with several other methods in its application to the analysis of a practical 15-story reinforced concrete building frame. The study was particularly useful in that the frame was irregular. Apparently, this investigation favorably impressed Mr. Reese with the usefulness of the simplified method as a time-saving device.

Mr. Floris seems to be rather vague as to the application of the simplified method of wind-stress analysis. Perhaps that is why he remains faithful to the classical method of simultaneous equations. He errs in classing the Cross

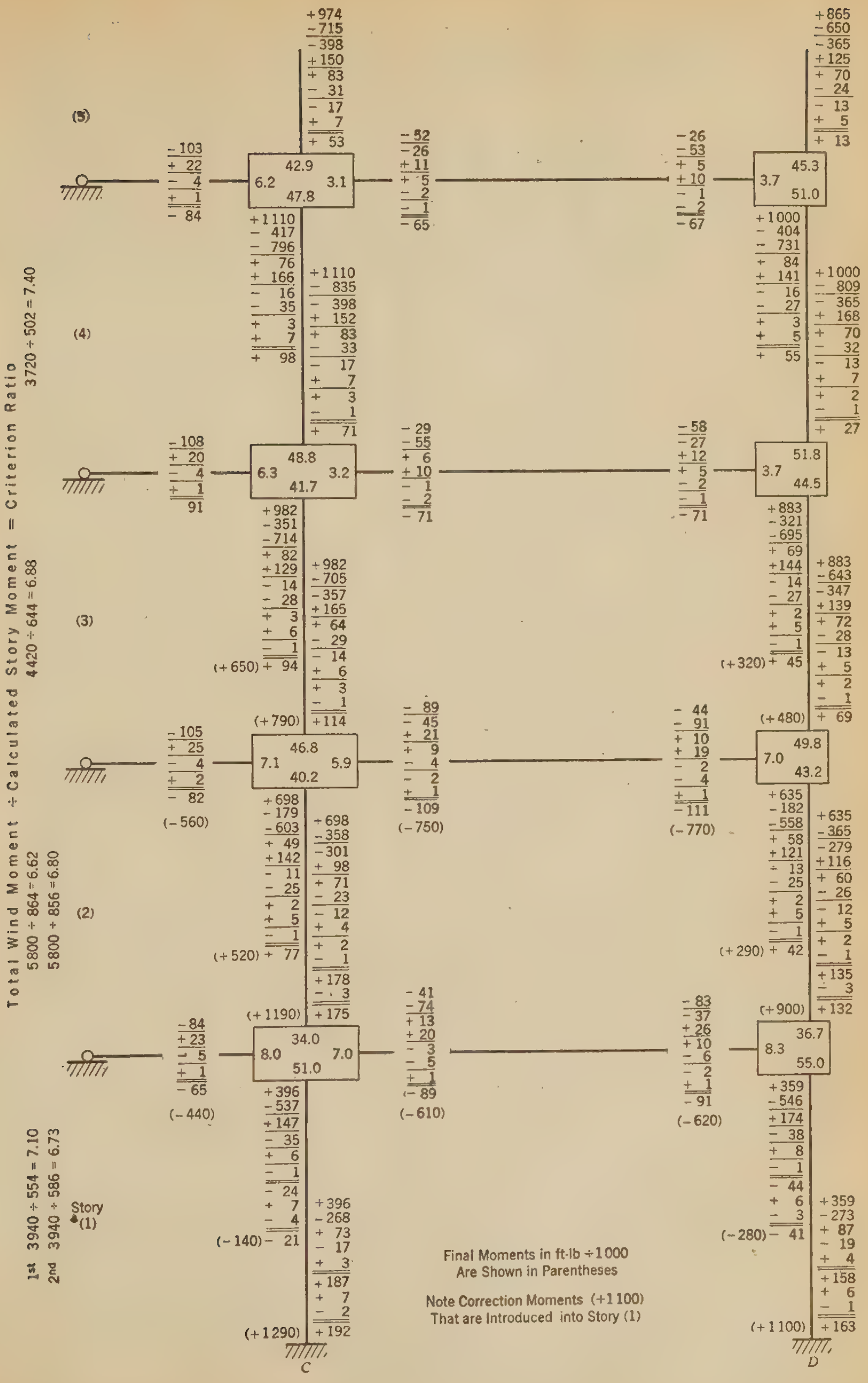
Total Wind Moment ÷ Calculated Story Moment = Criterion Ratio
3720 ÷ 502 = 7.40
4420 ÷ 644 = 6.88
5800 ÷ 864 = 6.62
5800 ÷ 856 = 6.80
1st 3940 ÷ 554 = 7.10
2nd 3940 ÷ 586 = 6.73
(5)
(4)
(3)
(2)
Story (1)
+974 −715 −398 +150 +83 −31 −17 +7 +53
−103 +22 −4 +1 −84
42.9 6.2 3.1 47.8
−52 −26 +11 +5 −2 −1 −65
−26 −53 +5 +10 −1 −2 −67
45.3 3.7 51.0
+865 −650 −365 +125 +70 −24 −13 +5 +13
+1110 −417 −796 +76 +166 −16 −35 +3 +7 +98
+1110 −835 −398 +152 +83 −33 −17 +7 +3 −1 +71
+1000 −404 −731 +84 +141 −16 −27 +3 +5 +55
+1000 −809 −365 +168 +70 −32 −13 +7 +2 −1 +27
−108 +20 −4 +1 91
48.8 6.3 3.2 41.7
−29 −55 +6 +10 −1 −2 −71
−58 −27 +12 +5 −2 −1 −71
51.8 3.7 44.5
+982 −351 −714 +82 +129 −14 −28 +3 +6 −1 +94 (+650)
+982 −705 −357 +165 +64 −29 −14 +6 +3 −1 +114 (+790)
+883 −321 −695 +69 +144 −14 −27 +2 +5 −1 +45 (+320)
+883 −643 −347 +139 +72 −28 −13 +5 +2 −1 +69 (+480)
−105 +25 −4 +2 −82 (−560)
46.8 7.1 5.9 40.2
−89 −45 +21 +9 −4 −2 +1 −109 (−750)
−44 −91 +10 +19 −2 −4 +1 −111 (−770)
49.8 7.0 43.2
+698 −179 −603 +49 +142 −11 −25 +2 +5 −1 +77 (+520)
+698 −358 −301 +98 +71 −23 −12 +4 +2 −1 +178 −3 +175 (+1190)
+635 −182 −558 +58 +121 −13 −25 +2 +5 −1 +42 (+290)
+635 −365 −279 +116 +60 −26 −12 +5 +2 −1 +135 −3 +132 (+900)
−84 +23 −5 +1 −65 (−440)
34.0 8.0 7.0 51.0
−41 −74 +13 +20 −3 −5 +1 −89 (−610)
−83 −37 +26 +10 −6 −2 +1 −91 (−620)
36.7 8.3 55.0
+396 −537 +147 −35 +6 −1 −24 +7 −4 −21 (−140)
+396 −268 +73 −17 +3 +187 +7 −2 +192 (+1290)
+359 −546 +174 −38 +8 −1 −44 +6 −3 −41 (−280)
+359 −273 +87 −19 +4 +158 +6 −1 +163 (+1100)
C
D
Final Moments in ft-lb ÷ 1000 Are Shown in Parentheses
Note Correction Moments (+1100) That are Introduced into Story (1)

FIG. 20.—ANALYSIS BY THE SIMPLIFIED METHOD OF A FRAME HAVING STIFF COLUMNS AND FLEXIBLE GIRDERS. PROPERTIES GIVEN IN FIG. 18, FIXED-END MOMENTS FROM TABLE 12.

TABLE 12.—COMPUTATIONS FOR ESTIMATING STORY DEFLECTIONS FOR FRAME HAVING STIFF COLUMNS AND FLEXIBLE GIRDERS. PROPERTIES GIVEN IN FIG. 18.

Story	Height, in feet	ΣK_c, in inches³	ΣK_g, in inches³	r	n	Moment division, percentage	Shear, in kips	Total moment, M_t, in kip-feet	Total column moments, in kip-feet	$E\theta \div 1\,000$	DEFLECTIONS, IN INCHES			FIXED-END COLUMN MOMENTS, IN INCH-POUNDS ÷ 100 000	
											$E\,\theta_{avg.} \times h \div 1\,000$	$E\,\delta \div 1\,000$	$E\,\Delta \div 1\,000$	A or D	B or C
						50									
6	12	472					300	3 600							
						50			1 800						
			30							121					
						50			1 830						
5	12	472					305	3 660			17 600	1 100	18 700	865	974
						50			1 830						
			30							123					
						50			1 860						
4	12	528					310	3 720			18 200	1 000	19 200	1 000	1 110
						50			1 860						
			30	0.061						130					
					0.222	46			2 030						
3	14	452					316	4 420			21 500	1 600	23 100	883	982
					0.261	54			2 390						
			42	0.100						126					
						50			2 900						
2	18	390					322	5 800			21 400	3 200	24 600	635	698
						50			2 900						
			58	0.119						72					
					∞	33*			1 310						
1	12	584					328	3 940			5 200	1 000	6 200	359	396
			∞	∞	2 ∞	67*			2 630						
										0					

* Moment division where first floor girder is relatively slender.

method of balancing moments as fundamentally less accurate than the slope deflection method, and he is further mistaken in assuming that the simplified method is tedious in practical cases, as has been shown by several discussers. Contrary to Mr. Floris' viewpoint, the writer feels that the importance of the elementary arithmetical procedure of the simplified method needs further emphasis.

Mr. Gedo is correct in emphasizing the importance of direct stress deformations on the moments in a Vierendeel truss, although the writer made no error in stating that direct stress deformations usually are neglected. The writer made an exhaustive study of this problem several years ago[35] and the conclusions reached were in agreement with Mr. Gedo's observations. Mr. Gedo might have saved himself considerable time by using the simplified method instead of the method of least work in solving his example. The effect of direct stress deformation is easily introduced into the analysis by the simplified method.

Mr. Skytte offers several rather complicated formulas for locating points of contraflexure and for determining girder shears in connection with a deflection estimate. The writer does not question the accuracy of such formulas, but wonders when they would be useful. The simple method of estimating deflections as given in the original paper has proved to be quite satisfactory; even rather startingly so. (See Figs. 18 and 21.) In fact, the writer would suggest that the method be simplified by omitting the use of the r and n-ratios, except in irregular stories as shown in Table 12. No practical case occurs to the writer in which greater precision would be neecssary.

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[35] Thesis presented to the Univ. of Illinois in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Engineering.

Mr. Richards is disturbed about the time consumed by the trial-and-error method for the design of a building frame. He suggests that in structural members, such as beams or columns, one proceeds directly to the selection of size and asks for a similar direct procedure for tall building design. The writer believes that this is equivalent to asking for the moon. A building frame is an indeterminate structure in which the moments are controlled by deflections. One must "have" a building before he can analyze it.

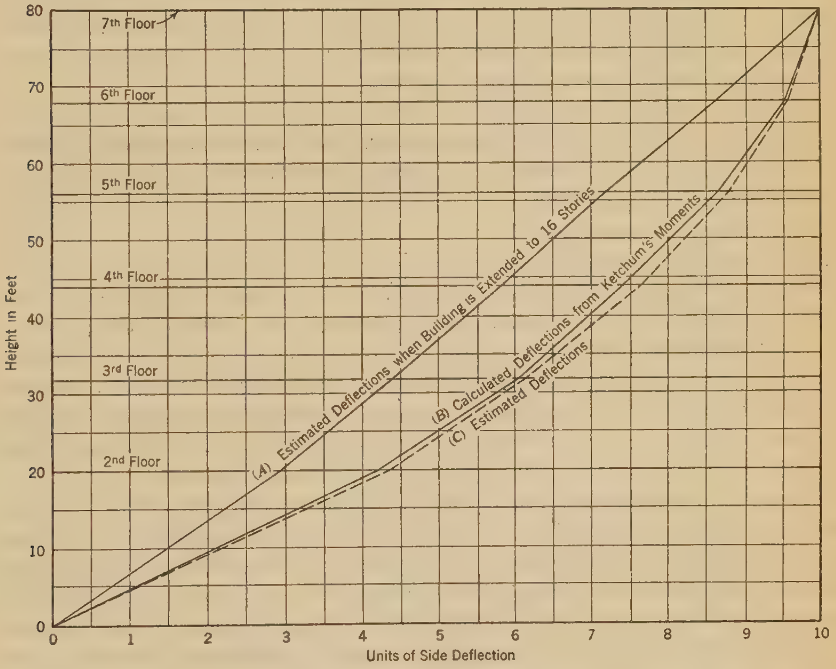

FIG. 21.—COMPARISON OF LATERAL DEFLECTIONS FOR THE KETCHUM FRAME.

There is a possible answer to Mr. Richard's question; namely, a building might be proportioned to justify the assumptions of a statically determinate method of analysis; then the true wind moments would be as assumed and the design would be completed in a single operation. The only difficulty here is that "the tail is wagging the dog." A method of analysis is set up on a pedestal, and the more important procedure of design is relegated to the background. The designer can study economical proportions only through a limited range and a truly economical structure is seldom possible. It is surely wasteful always to force points of contraflexure to the centers of members, and to attempt to suggest to the structure which members shall carry direct stress, approaches the impossible unless the frame is very regular.

There seems to be a popular fetishism to this ideal of design, but the writer doubts that it will survive or even that it can be strictly applied to an irregular frame.

Like most extremely practical men, Mr. Spurr has so frequently encountered difficulties produced by structural details that he may perhaps tend to under-emphasize the usefulness of a theoretical tool of analysis just as a college professor may tend to over-emphasize its importance. When one is faced with the daily realization that angles, split beams, and knee-braces must be used more or less indiscriminately, and that all are commonly assumed to produce rigid connections when the structure is analyzed (which, of course, is not true) the assumption of points of contraflexure at the mid-height of columns takes on a less serious aspect. However, despite these disturbing factors, the writer prefers to place his faith in a method of analysis that takes elastic deflections into account.

Mr. Spurr errs in stating that the author has not established a method of caring for the effect of column deformations. The matter receives full attention under the heading, "Effect of Column Deformation." The evaluation of the effect of column deformation is not difficult when a second analysis follows revision of proportions.

Mr. Fleming supports, at least partly, the writer's contention that the method of successive corrections, of which the simplified method is one variation, is "feasible" for application to practical tall building design where the slope deflection method and the other classical methods were beyond reasonable use. The writer considers this particularly significant, because Mr. Fleming's statically determinate methods are those that careless designers are likely to try to use on irregular structures, whereas Mr. Fleming himself states that irregular structures "should receive special study."

The six-story bent analyzed by M. S. Ketchum, M. Am. Soc. C. E., is mentioned by Mr. Fleming[25] as showing a variation of criterion ratios from 5 to 50% when analyzed by the simple method of balancing K-values. His statement is correct, but perhaps misleading to some readers. No different result could be expected. The balancing of K-values is based on the assumption of a straight-line (or gradually curved) deflection diagram. The Ketchum bent shows a variation of 20 to 1 between the total moments in the first and sixth stories, while the K-values of the columns are scarcely doubled and the girders are unchanged. A tall building, on the other hand, shows a corresponding increase in K-values and story moments from top to bottom. The tall building will commonly approach a straight-line deflection curve while that for the Ketchum bent will be roughly parabolic.

The curves of Fig. 21 are deflection diagrams for the Ketchum bent. The two similar curves (B and C) represent the deflection of the six-story bent as estimated by the writer's method and as calculated from the Ketchum moments. The agreement between these curves shows that the moments obtained by the simplified method (based on the estimated deflections) would check the Ketchum moments closely. The third curve (A) approaches a straight line and is the estimated deflection diagram for the lower six stories of Professor Ketchum's bent if it is extended upward to form a 16-story building. Note that for this practical case the deflection curve does approach a straight line and the moments could be obtained within a range of error of 10% by following the procedure of balancing *K*-values, as illustrated for the American Insurance Union Building, in Fig. 5. Correction moments would be required in the lower story only. Incidentally, it seems that the idea of introducing correction moments made little impression on the discussers. Experience will show that this is one of the most valuable tools of the simplified method.

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TRANSACTIONS

Paper No. 1871

EVAPORATION FROM WATER SURFACES
A SYMPOSIUM

PAGE

Evaporation from Different Types of Pans.
By Carl Rohwer, Assoc. M. Am. Soc. C. E. 673

Evaporation from Reservoir Surfaces.
By Robert Follansbee, M. Am. Soc. C. E. 704

Standard Equipment for Evaporation Stations: Final Report of Sub-Committee on Evaporation of the Special Committee on Irrigation Hydraulics 716

With Discussion by Messrs. Ralph R. Randell, C. E. Grunsky, Charles H. Lee, S. T. Harding, L. Standish Hall, Adolpho Santos, Jr., John W. Pritchett, Ivan E. Houk, R. I. Meeker, Carl Rohwer, Robert Follansbee, and Sub-Committee on Evaporation, Special Committee on Irrigation Hydraulics.

Note.—Published in February, 1933, *Proceedings*.

FOREWORD

These two papers and the recommended list of standard equipment constitute the Final Report of the Special Committee on Irrigation Hydraulics on the subject of evaporation. The Symposium has been developed since the submission of the work done by Mr. Rohwer, which was accepted for publication as a separate paper. The official report of the Sub-Committee on Evaporation, covering only the subject of standard equipment, was next added; and, finally, Mr. Follansbee's contribution was accepted to round out the entire subject.

The Committee has had as one of its subjects for investigation that of Evaporation from Reservoirs. One of the primary objects of that work has been to find a suitable set of coefficients by which the great variety of existing data on evaporation could be corrected, to the end that evaporation from a large water surface could be expressed in terms of that from the various types of pans in use.

The first objective has been fulfilled in the paper by Mr. Rohwer and the second by the report of the Sub-Committee on Evaporation. Mr. Follansbee's paper serves the purpose of applying the findings regarding pan evaporation to reservoirs. A great mass of detailed information, comprising 200 or more tables, with monthly evaporation records, has been placed on file by Mr. Follansbee, at Engineering Societies Library, 29 West 39th Street, New York, N. Y. These records contain data pertaining to evaporation from reservoirs all over the United States and in many other parts of the world.

The Special Committee on Irrigation Hydraulics has approved the report of the Sub-Committee on Evaporation unanimously and, further, recommends that these standards be followed wherever new evaporation stations are to be installed or old ones re-equipped.

By the Special Committee on Irrigation Hydraulics,

D. C. Henny, *Chairman,*
J. C. Stevens, *Secretary,*
B. A. Etcheverry
George W. Hawley
Ivan E. Houk
Morrough P. O'Brien
R. L. Parshall
J. L. Savage
Fred C. Scobey
I. C. Steele
Franklin Thomas

January 18, 1933.

EVAPORATION FROM DIFFERENT TYPES OF PANS

By Carl Rohwer,[1] Assoc. M. Am. Soc. C. E.

Synopsis

The rate of evaporation from different types of pans varies widely and before the evaporation data can be used in estimating the loss from large bodies of water, it is necessary to know the ratio between the evaporation from the type of pan used and that from a large water surface, or the relation between the evaporation from the pan and some pan for which the ratio of the evaporation to that from a large water surface is known.

A summary of the results of the available records, where a comparison has been made between the evaporation from different types of pans or between the evaporation from a pan and a large water surface under similar conditions, is given in this paper, together with recommendations as to the best types of pan to use under different conditions, and the procedure to be followed in taking the observations.

Introduction

Available records of evaporation from pans of different types, or from pans and large water surfaces under similar conditions, are either scattered through many publications or are buried among the unpublished records of various organizations. The writer has assembled such data and re-arranged them for presentation in tabular form. Credit is given in every instance to the publication or agency from which the record was obtained. The tables are published as an Appendix to the paper.

Comparative Results

The records of the comparisons between the evaporation from many types of pans were taken (see Appendix), but particular attention was given to the pans most commonly used, such as the United States Weather Bureau Class *A* land pan and floating pan, the Colorado sunken pan, the United States Geological Survey floating pan, and the United States Bureau of Plant Industry sunken pan. Comparisons were also made, where records had been taken, between the evaporation from these pans and from large bodies of water. In addition to the record of the evaporation, all meteorological data pertaining to each tank were included. Where possible the data at the station were used, but when no records had been kept, the data were taken from the records of the nearest Weather Bureau Station.

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[1] Associate Irrig. Engr., Colorado Experiment Station, Fort Collins, Colo.

All the records have been summarized and only the mean data are included in this paper. In some cases merely the means for the entire period of the observations are given, but where sufficient records are available the monthly or the daily means are reported. The daily means are given only in those cases where intensive studies of evaporation were made for short periods, and where the observations were taken at frequent intervals during the day.

None of the records has been eliminated, nor have any observations been discarded, except where the original records indicate that the observations are in error. It was assumed that the observations were correct, except as noted, and that differences in the results were due to variations in the setting or the conditions under which the pans were exposed. Consideration was given, however, to the fact that the records from floating pans are less reliable than those from land pans on account of the difficulty in making accurate readings of the evaporation loss, and also because during wind storms water frequently splashes into the pan from the reservoir, or splashes out of the pan when there is a pronounced swell after a storm.

The comparisons of evaporation from pans and from large water surfaces are few and the results are uncertain because of the difficulty in segregating the evaporation from the other losses. Experiments on the evaporation from pans of different sizes, however, show that the size of the pan has a proportionately less effect on the evaporation as the size of pan increases. This fact is used as the basis for determining the relation between the evaporation from pans and from large water surfaces. The comparative data on the evaporation from reservoirs and from pans similarly exposed are presented to show how closely the facts are in accord with the theory, and also to give some idea as to the variation that may be expected in the results.

In reporting the data all the known details concerning the setting of the pans and the conditions under which they operated, are included. Unfortunately, many details are frequently lacking in the reported data. Under these circumstances, if there are discrepancies in the results the reasons for them can only be conjectured.

The tables giving these data are arranged according to the types of pans. All tables showing the comparison of one type of pan with various other types are grouped together. From the mean values taken from these tables, a summary of all the results in which a comparison with any of the standard pans was possible, was prepared. These mean ratios are not the means of the monthly ratios, but are the means of the ratios of the total evaporation losses from the pans.

The experiments[2] made by the late R. B. Sleight, Assoc. M. Am. Soc. C. E., at Denver, Colo., during 1915, 1916, and 1917, cover a wide range of conditions, and the results obtained are uniformly consistent. In using the ratios for the floating tanks, however, it should be kept in mind that the floating pans were not placed at the same point as the land pans and that results similar to those obtained elsewhere can not be expected. The obser-

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[2] *Journal of Agricultural Research,* Vol. X, No. 5, July 30, 1917.

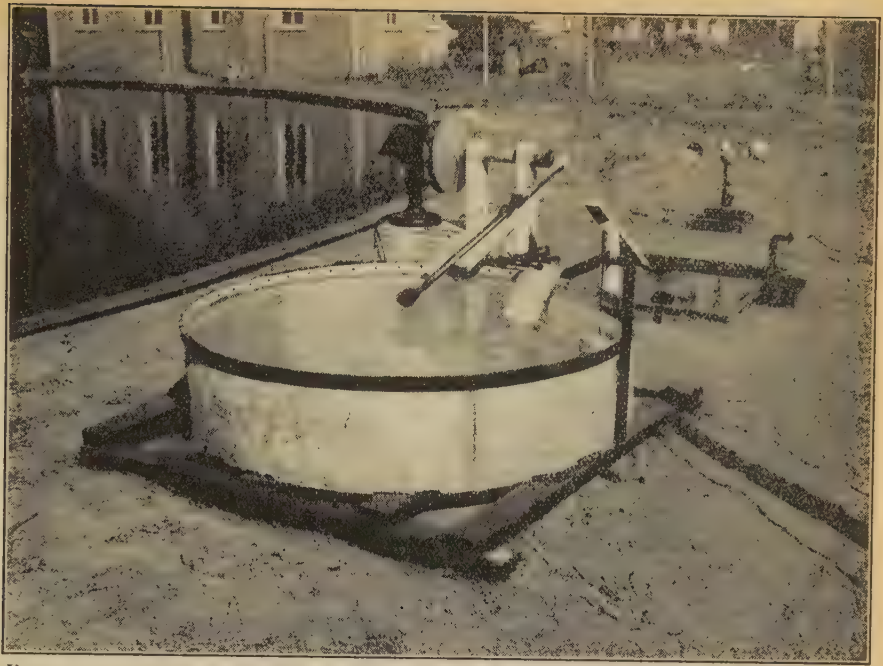

Fig. 1.—U. S. Weather Bureau Class A Land Pan, Colorado Sunken Pan, and 86-Foot Circular Reservoir and Auxiliary Equipment at Fort Collins, Colo.

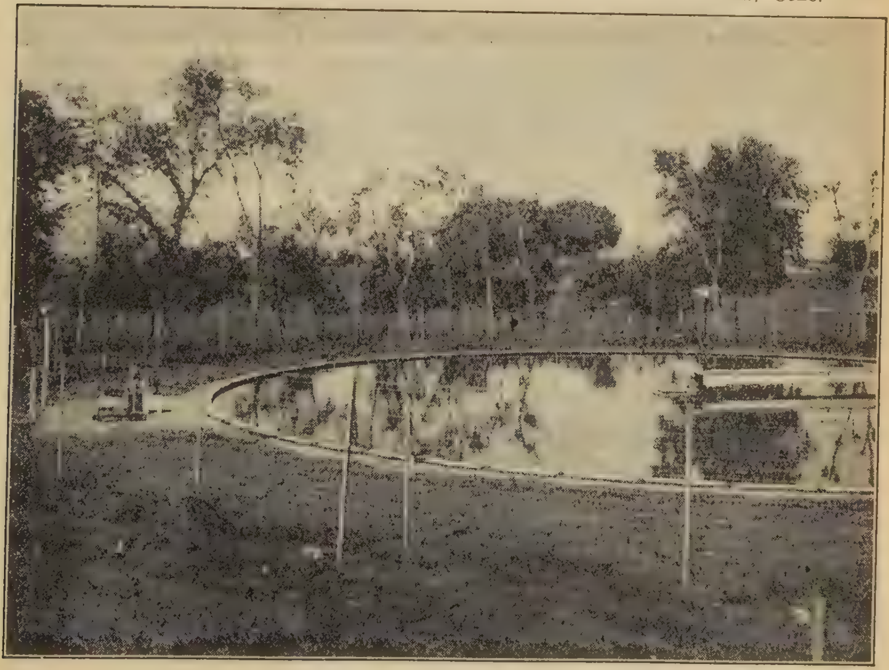

Fig. 2.—Arrangement of Equipment for Conducting Experiments on the 85-Foot Circular Reservoir and Various Standard Pans at Fort Collins, Colo.

vations of floating pans may be compared with each other, however, as they were all subjected to the same conditions.

The comparisons of the evaporation from different types of pans with that from the 85-ft circular reservoir at Fort Collins, Colo., are the results of a large number of observations made under carefully observed conditions. The observations cover three seasons. During the first two seasons, the readings were taken four times daily at 6-hr intervals, and during the last season they were taken three times daily. The 85-ft reservoir used in making the comparison was lined with copper to insure water-tightness. Carefully calibrated instruments were used in taking the observations. It is believed that the results of these comparisons are representative of the conditions under which they were taken. The equipment used in making the study is shown in Figs. 1 and 2. In Fig. 2, the Geological Survey floating pan is shown in the center of the reservoir.

The evaporation from East Park Reservoir and from the various evaporation pans at Stonyford, Calif., was compared under conditions that were favorable for obtaining reliable results. East Park Reservoir is in a practically water-tight basin, and during the period of the tests there was no precipitation and practically no inflow or outflow. Evaporation readings were taken twice daily. The results obtained are in accord with those found at Fort Collins and Denver, except in the case of the floating pan at Denver, and as previously stated, the land and floating pans at Denver were some distance apart. Figs. 3 and 4 show the evaporation pans at East Park Lake.

The ratios of the evaporation from the Weather Bureau Class *A* land pan and the Colorado sunken pan to the evaporation from the 12-ft circular sunken pan at Denver, the 85-ft circular reservoir at Fort Collins, and the 1 800-acre reservoir at Stonyford, are almost identical. Therefore, it is evident that between these limits (surfaces of 113 sq ft and 1 800 acres), there is no decrease in the evaporation per unit area as the size of the water surface increases. These conclusions are confirmed by experiments at Milford, Utah, on the evaporation from a Weather Bureau Class *A* land pan and a 12-ft circular sunken pan. In view of the fact that these observations were made at widely separated points, it may be assumed that these ratios are constant. If these conclusions are correct, it is possible to compute the evaporation from a lake or reservoir when the evaporation from any type of pan for which the ratio has been determined, is known.

OTHER RESERVOIR EVAPORATION COMPARISONS

A comparison of the evaporation from a buried pan and the total loss from Newell Reservoir, Alberta, Canada,[3] by the Department of Natural Resources of the Canadian Pacific Railroad for the period, 1919 to 1925, shows that on the average the total loss from the reservoir is 95% of that from the land pan. The season ratios, however, vary from a minimum of 77% to a maximum of 116 per cent. This large variation is probably due to the fact that the loss from the reservoir was determined from the seasonal inflow, outflow, precipitation, and change in elevation. Errors in any of

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[3] *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 350.

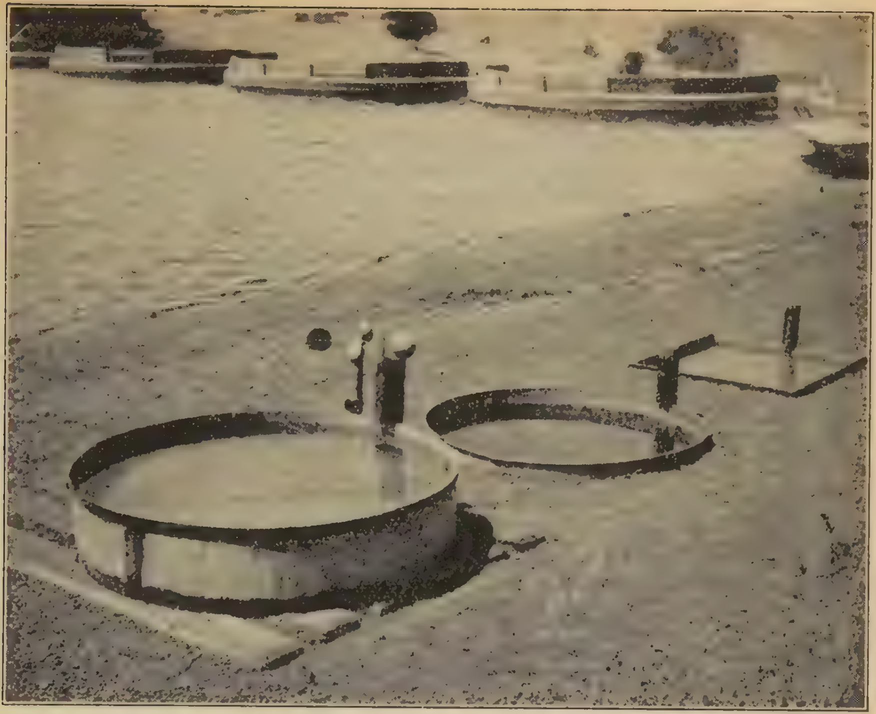

FIG. 3.—U. S. WEATHER BUREAU CLASS *A* LAND PAN, 4-FOOT CIRCULAR SUNKEN PAN, AND COLORADO PAN, AT EAST PARK RESERVOIR, STONYFORD, CALIF.

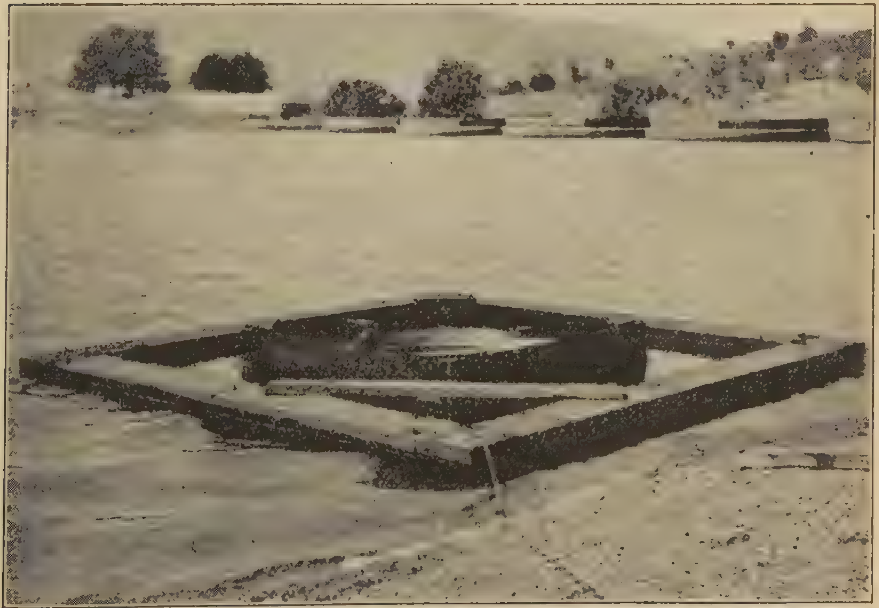

FIG. 4.—U. S. WEATHER BUREAU CLASS *A* LAND PAN USED AS A FLOATING PAN AT EAST PARK RESERVOIR, STONYFORD, CALIF.

these factors would be charged to evaporation. Newell Reservoir has an area of 16 430 acres at Elevation 2 485, and an available capacity of 189 000 acre-ft. The evaporation pan was 4 ft in diameter and 2 ft deep. It was sunk in the ground to within 2 in. of the top, and the water was maintained at the ground level. The pan was located near the shore of the reservoir.

Experiments during 1904 and 1905 by Duryea[4] on the evaporation from land and floating pans in the Santa Clara Valley, California, show that the average evaporation from the floating pans is 71% of that from the land pan. This value is based on the average results from six land pans and three floating pans divided into two groups. The values of the relation varied from a minimum of 55% to a maximum of 85 per cent. All pans were 3 ft square and 12 in. deep and the depth of water was approximately 10 in. The land pans were embedded in the ground and banked with earth nearly to their rims. The floating pans were floated on the Laguna Seca and, apparently, were freely exposed to the air.

Experiments were reported by S. T. Harding,[5] M. Am. Soc. C. E., on the evaporation from Lake Elsinore, in California, and from a pan 3 ft square floating in the lake for the period from May to September, 1916. The results show that the evaporation from the lake is 84% of that from the pan, but that the relation by months varies from a minimum of 63% to a maximum of 114 per cent. The evaporation from the lake was determined from the record of the inflow, outflow, precipitation, and change of stage. The high-water area of Lake Elsinore is 5 500 acres and the elevation, 1 261 ft.

Experiments reported[6] by J. B. Lippincott, M. Am. Soc. C. E., on the evaporation from Lake Hodges and Cuyamaca Lake, and the evaporation from floating pans and land pans are contradictory. The evaporation from Lake Hodges (area, 1 317 acres, and elevation, 315 ft), during six months from June, 1919, to October, 1921, when there was no draft from the lake, was 96% of the evaporation from a sunken pan 3 ft square and 18 in. deep, sunk with the rim flush with the ground and filled with water to within 4 in. of the top. The evaporation from Cuyamaca Lake (area, 978 acres, and elevation, 4 600 ft) during November, 1916, and January, 1919, when there was no draft from the lake, was 108% of that from a floating pan 3 ft square and 18 in. deep, filled with water to within 4 in. of the top. Later, the pan was sunk in the ground and under these conditions, during November, 1921, and June, 1922, the evaporation from the lake was 119% of the pan evaporation. Cuyamaca Lake is surrounded by a large swamp area, and the vegetation in this area may have increased the loss, due to the transpiration from the plants.

Choice of Type of Evaporation Pan

The type of pan chosen for evaporation observations will depend on the purpose of the experiment and on a knowledge of the information available for the different types of evaporation pans. Although a pan may be desirable from several standpoints, it should not be chosen unless the relation of the evaporation from the pan to other types of pans and to large water surfaces is thoroughly established.

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[4] *Engineering News,* Vol. 67, pp. 380–383.

[5] San Jacinto River Hydrographic Investigations, 1922, Div. of Water Rights, State of California, *Bulletin 9,* State Dept. of Eng.

[6] *Transactions,* Am. Soc. C. E., Vol. 90 (June, 1927), pp. 356–359.

The types of pans most commonly used in the United States are: (1) The U. S. Weather Bureau Class *A* land pan shown in Figs. 2, 3, and 4; (2) the U. S. Bureau of Plant Industry sunken pan; (3) the Colorado sunken pan (see Figs. 2 and 3); (4) the U. S. Geological Survey[7] floating pan; and (5) the U. S. Weather Bureau floating pan. Each of these pans is built according to definite specifications and has certain advantages and disadvantages. For convenience, these types will be referred to by number in this paper.

Pan (1).—The Class A Land Pan of the United States Weather Bureau.—The Class *A* land pan is used more than any other type, and for general evaporation studies is probably the most satisfactory. This is the standard pan of the U. S. Weather Bureau and is the type used at all its evaporation stations. Other agencies also frequently install this type of pan and follow the Weather Bureau procedure in taking the observations. As shown in Fig. 5, this pan is 4 ft in diameter and 10 in. deep. It is made of 22-gauge galvanized iron and is supported on a grillage of timbers so that the bottom of the tank is 6 in. above the original ground surface. The pan is filled with water to within 2 in. of the top and is refilled as soon as the water has dropped 1 in. below this elevation. A special micrometer hook-gauge is used to measure the evaporation. This gauge is located in a stilling-well which acts as a support for the gauge. The auxiliary equipment consists of an anemometer, a rain gauge, "maximum-and-minimum" thermometers, and an instrument shelter. The readings are taken twice daily at approximately 7:00 A. M. and 7:00 P. M.

The pan is simple in design and easy to operate. Being above the ground, it does not blow full of water in wind storms. The water does not splash into it and the snow does not drift into it when rain or snow is falling. The water in the pan is fully exposed to the air, and follows the air temperatures quite closely. As a result, the evaporation from it changes more quickly with the changes in temperature than from other types of pan. In some experimental work, this is a desirable feature, but the rapid change in the temperature makes it difficult to obtain the true mean temperature of the water.

From the standpoint of making comparisons with known data on evaporation, this type of pan is probably best adapted to the study of evaporation from water surfaces, because a large number of these pans have been installed in the United States, and many observational data obtained under similar conditions with a uniform procedure, are available from them. It is to be regretted that relatively few of these pans have been installed in the eastern part of the United States, and, unfortunately, all the data necessary for making a comparison between evaporation records at different stations are

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[7] In a letter dated December 11, 1929, N. C. Grover, M. Am. Soc. C. E., Chief Hydraulic Engineer of the U. S. Geological Survey, states that there is no official U. S. Geological Survey floating pan. The pan by that name was designed by E. F. Kriegsman, Assoc. M. Am. Soc. C. E.

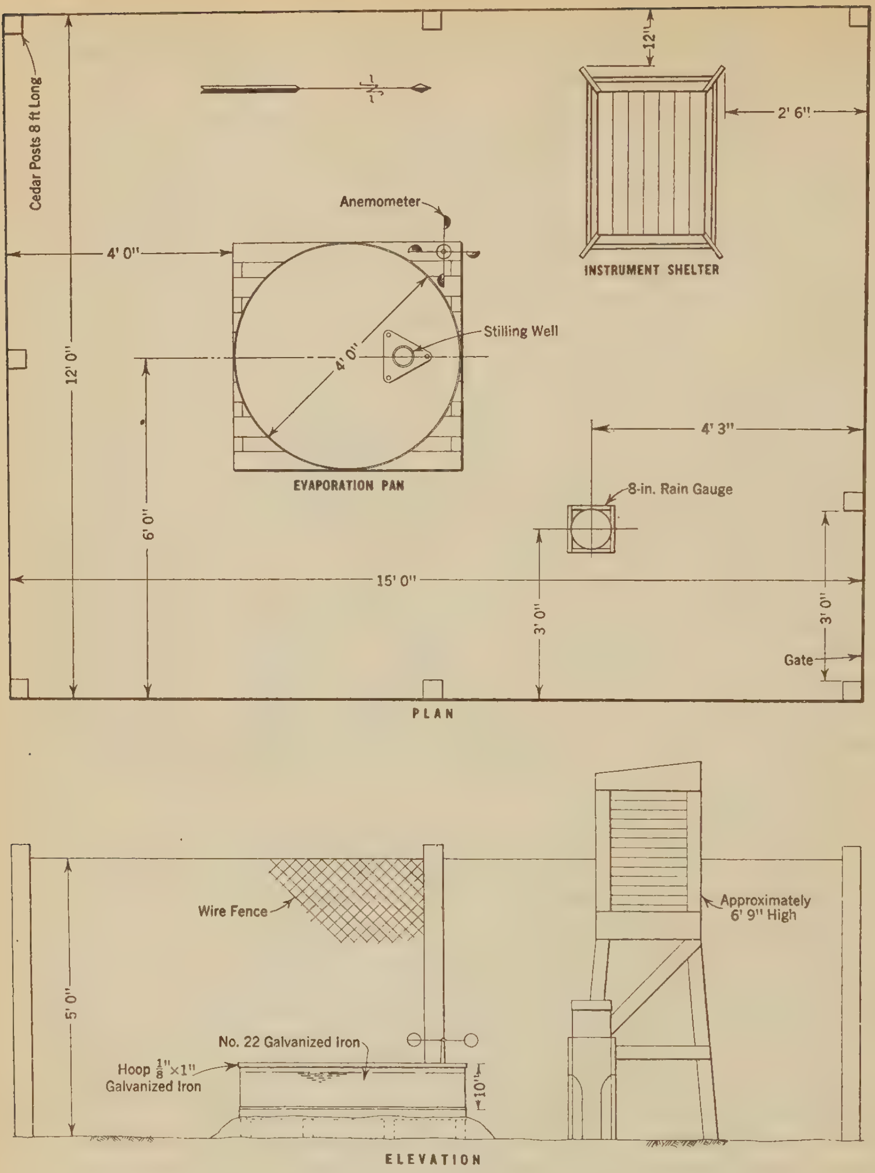

Cedar Posts 8 ft Long
12' 0"
4' 0"
Anemometer
Stilling Well
4' 0"
EVAPORATION PAN
12"
2' 6"
INSTRUMENT SHELTER
4' 3"
8-in. Rain Gauge
6' 0"
15' 0"
3' 0"
3' 0"
Gate
PLAN
Wire Fence
Approximately 6' 9" High
5' 0"
No. 22 Galvanized Iron
Hoop $\frac{1}{8}$"×1" Galvanized Iron
10"
ELEVATION

FIG. 5.—PLAN OF U. S. WEATHER BUREAU CLASS *A*, LAND PAN (TYPE (1)).

not observed by the Weather Bureau. The missing information, however, can usually be obtained from records of near-by Weather Bureau Stations.

Another criticism of this type is that the rate of evaporation is much higher than that from a large water surface under similar conditions, due to the fact that the pan is more fully exposed to the air than the reservoir. For this reason a large correction must be made in determining the reservoir evaporation from the pan record. This factor has been determined, however, under a wide variety of conditions, with very consistent results. Comparisons indicate that a factor of 0.69 or 0.70 should be used in converting the evaporation from the pan to that from the reservoir.

Pan (2).—United States Bureau of Plant Industry Sunken Pan.—The pan shown in Fig. 6 is also quite satisfactory. It is 6 ft in diameter and 2 ft. deep, is made of 22-gauge galvanized iron, and is sunk in the ground so that 4 in. of the rim projects above the ground surface. The water surface is maintained at approximately the elevation of the surrounding ground, and whenever it deviates more than $\frac{1}{2}$ in. from the level, due to either evaporation or precipitation, it is brought back to the standard elevation by adding or removing water. The evaporation from the pan is measured by a special micrometer point-gauge, placed on the stilling-well, which is attached to the outside of the pan. A standard rain gauge, anemometer, and instrument shelter of the Weather Bureau type are a part of each station. Maximum-and-minimum thermometers and a psychrometer are provided for determining the temperature and the relative humidity. The readings are taken daily.

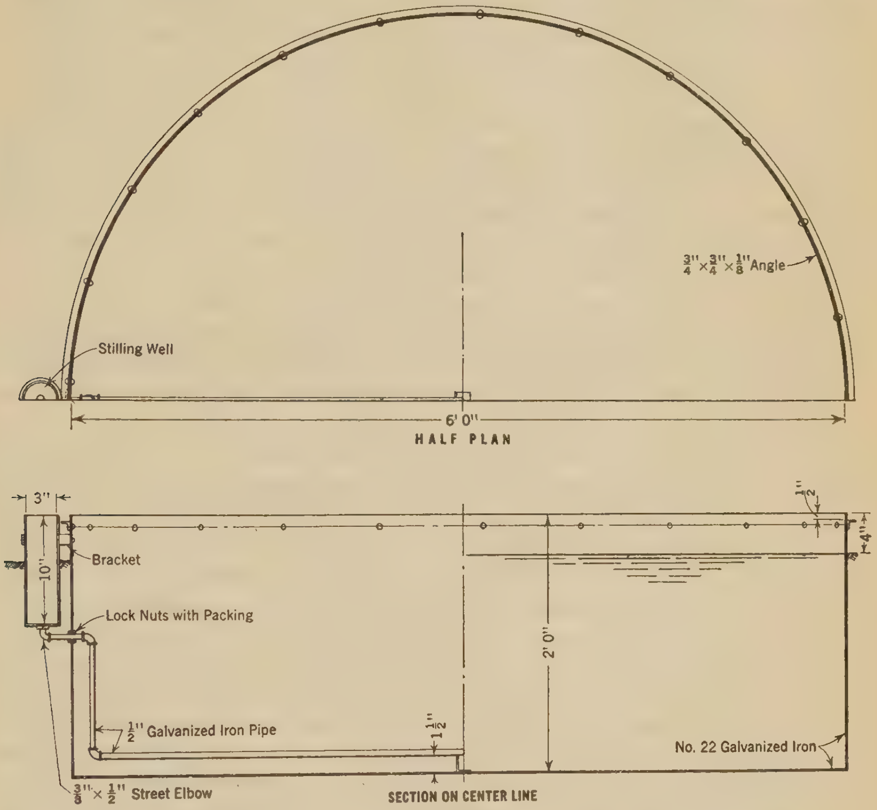

3/4"×3/4"×1/8" Angle
Stilling Well
6' 0"
HALF PLAN
3"
1/2"
4"
Bracket
10"
Lock Nuts with Packing
2' 0"
1/2" Galvanized Iron Pipe
1/2"
No. 22 Galvanized Iron
3/8"×1/2" Street Elbow
SECTION ON CENTER LINE

FIG. 6.—U. S. BUREAU OF PLANT INDUSTRY LAND PAN (TYPE (2)).

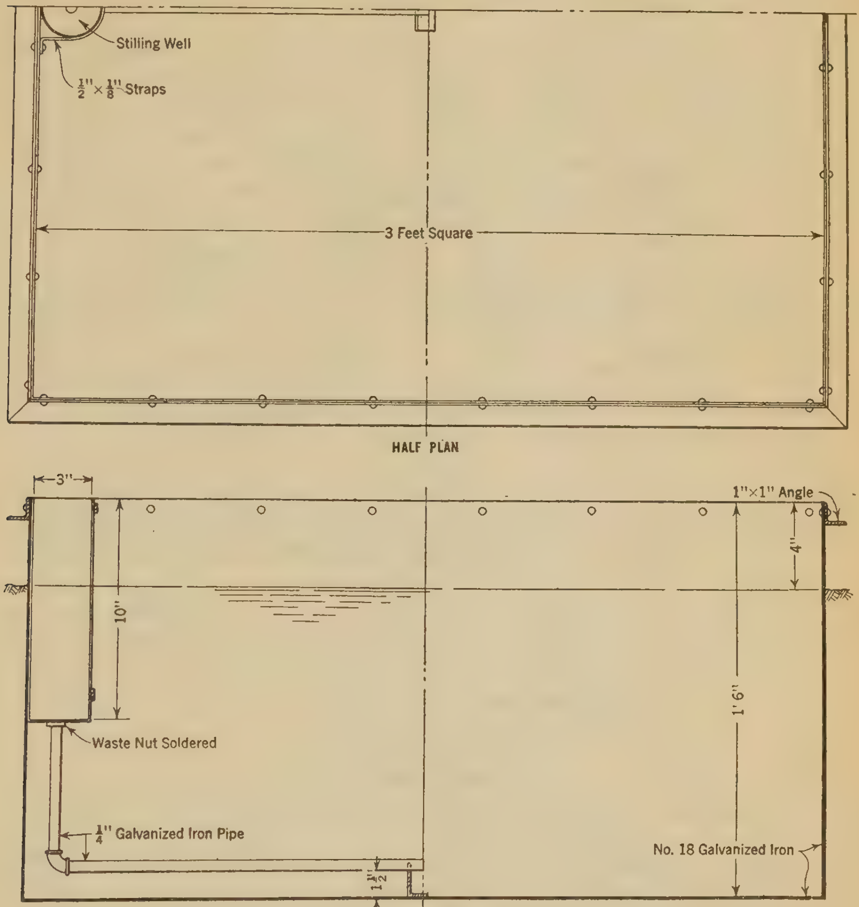
The Bureau of Plant Industry maintains an evaporation pan of this type at each of its dry land stations in the western half of the United States. Evaporation records, including complete meteorological data, are available at each of the stations up to and including 1916. After this date the water temperature records are lacking. The evaporation from this pan does not vary through as wide a range as that from Pan (1), nor does the temperature of the water follow the air temperature so closely. Since the variation in temperature is less, it is easier to determine the mean temperature. The evaporation is more nearly equal to that from a large water surface, and, consequently, a smaller correction is necessary to determine the evaporation from a large water surface. No direct comparisons are available between the evaporation from Pan (2) and from a large water surface, but comparisons have been made with pans for which this relation is known, and from this information it is possible to determine the factor to be used in computing the reservoir evaporation from the pan evaporation. This factor is given as 0.94 in Table 6, which is discussed subsequently.

Pan (2) is larger and more expensive to build and install than Pan (1) and data are not available from as many localities. Neither is the pan so well protected from splashing rain, drifting snow, and blowing dust and trash, as the Weather Bureau pan, but the fact that the rim is 4 in. above the ground minimizes the effect of these disturbing factors.

Pan (3).—The Colorado Sunken Pan.—The Colorado pan was developed at the Colorado Experiment Station. This pan, shown in Fig. 7, is usually made 3 ft deep, but will give satisfactory results if it is only 18 in. deep. It is made of 18-gauge galvanized iron and is sunk in the ground to within from 2 to 6 in. of the top, depending on conditions. Probably the most satisfactory depth is 4 in., because it tends to eliminate the difficulties due to splashing rain, drifting snow, and the accumulation of dust and trash to a considerable extent without introducing too great a rim effect due to the exposed metal. The water is maintained at the ground level and should not be permitted to deviate more than an inch each way from this level. A hook-gauge is used for measuring the evaporation and a stilling-well should be installed when accurate results are desired. A standard anemometer and rain gauge are provided for measuring the wind velocity and precipitation, and maximum-and-minimum thermometers for the temperature. A sling psychrometer is used to determine the humidity. Readings on the pan are taken twice daily.

The Colorado pan has not been used as extensively as Pans (1) and (2). It has advantages, however, which warrant a wider use of this type. It is cheap and easy to build and install. Due to the fact that the pan is sunk in the ground, the water temperature lags behind the air temperature. In this respect it resembles a large water surface more nearly than Type (1), and is similar to Pan (2). For this reason the mean temperature of the water is easy to determine. The evaporation from the Colorado pan is almost identical with that from a floating pan of the same size and shape, exposed under the same meteorological conditions, and is much easier to maintain.

The correction factor, 0.78 (in Table 6), for computing the equivalent evaporation from a reservoir is between that for Pans (1) and (2). It has been determined under a wide range of conditions with uniformly consistent results.


Stilling Well
½"×⅛" Straps
3 Feet Square
3"
1"×1" Angle
4"
10"
1' 6"
Waste Nut Soldered
¼" Galvanized Iron Pipe
No. 18 Galvanized Iron
1½"

HALF PLAN

SECTION ON CENTER LINE

FIG. 7.—COLORADO LAND PAN (TYPE (3)).

Pan (4).—The United States Geological Survey Floating Pan.—The evaporation pan in Fig. 8, was designed for use in measuring the loss from lakes and reservoirs because it was thought that evaporation from a floating pan would be identical with that from a reservoir, but the experimental results have not substantiated this conclusion. This pan is made of 18-gauge galvanized iron and is supported by two cylindrical metal tubes, so that it floats in the water with 3 in. of the rim above the surface. The water inside the tank is kept at the same depth as that outside. Surging in the pan is reduced by two diagonal diaphragms beneath the water surface, which are perforated with 1-in. holes as shown. In order to protect it from the waves, the pan is surrounded by a raft supported by barrels to keep it from sinking when water-logged. Evaporation is determined by measuring the water required to bring the level to a fixed point which is attached to the intersection of the diaphragms. A special cup holding exactly enough water to raise the level of the pan 0.01 in. is used in making the measurement. When complete meteorological records are desired, the same instruments should be used as are provided for Pan (2).

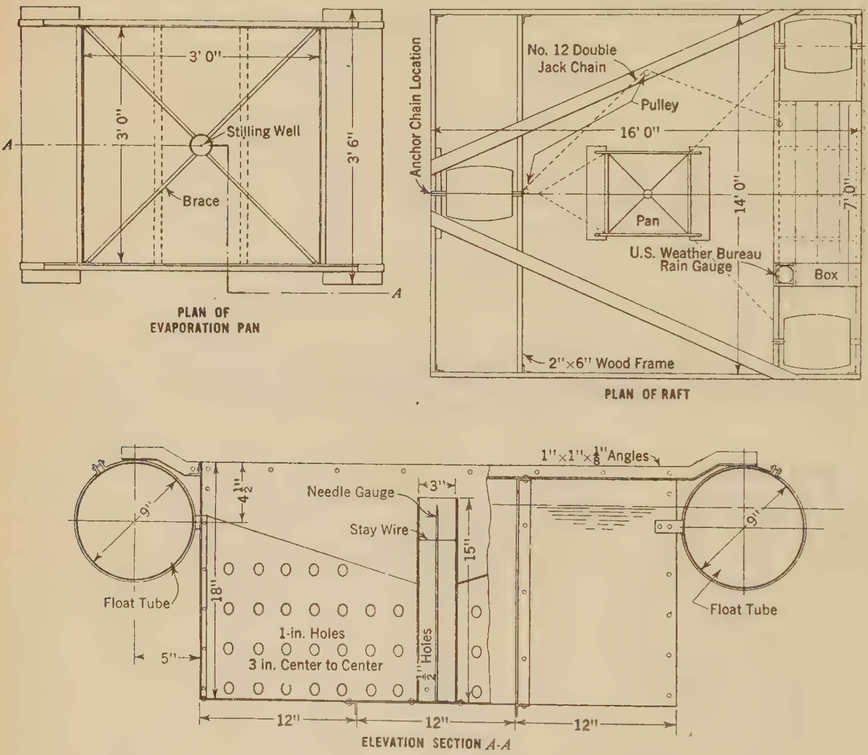

3' 0"
3' 0"
3' 6"
A
A
Stilling Well
Brace
PLAN OF EVAPORATION PAN
Anchor Chain Location
No. 12 Double Jack Chain
Pulley
16' 0"
14' 0"
7' 0"
Pan
U.S. Weather Bureau Rain Gauge
Box
2"×6" Wood Frame
PLAN OF RAFT
1"×1"×⅛"Angles
9"
9"
4½"
Needle Gauge
3"
Stay Wire
15"
18"
Float Tube
Float Tube
5"
1-in. Holes
3 in. Center to Center
½" Holes
12"
12"
12"
ELEVATION SECTION A-A

FIG. 8.—PLAN AND SECTION, U. S. GEOLOGICAL SURVEY FLOATING PAN (TYPE (4))

Being in the water, Pan (4) is protected from drifting snow and is not affected by the splashing of the rain, because as much splashes in as splashes out. Neither dust nor trash can blow into the pan. It is subject to the same conditions as those that occur in the lake or reservoir, and, consequently, the factor for computing the equivalent evaporation from the large water surface should be nearer unity than the factors for the land pans. The experimental results are inconsistent; however, the mean value of the factor is 0.77. The evaporation from the floating pan is almost identical with that from a similar sunken pan (see Table 3(*a*) and Table 6). Since, in comparison with the land pan (Type (1)), Pan (4) is more expensive to construct, more difficult to maintain, and is constantly subject to the danger of being splashed full of water from outside waves or being partly emptied by water splashing out of the tank, due to the rolling of the pan, this type is losing favor rapidly and is being replaced by some type of land pan.

Pan (5).—United States Weather Bureau Floating Pan.—Pans of the Weather Bureau type, as shown in Fig. 4, are sometimes used as floating pans. They have all the advantages and disadvantages of Pan (4), except that they roll more when there is a swell because of their greater size. This condition has been overcome to some extent by building cellular baffles in the pan below the water surface. Evaporation is almost identical with that from a buried pan of similar shape (see Table 1). The constant for converting the pan evaporation to reservoir evaporation has been established as 0.78 (Table 6). Equipment similar to that used at stations of the Bureau of Plant Industry is recommended.

From the foregoing discussion and from the experimental results, Pan (1) seems to be best adapted to the study of ordinary evaporation problems. For special problems, some of the other types of pans may give better results and, under these conditions, the merits of the different pans for the special problem should be investigated. The evaporation from Pan (2) probably approaches that from a large body of water more nearly than any of the other standard pans, but the factor for converting the pan evaporation to the reservoir evaporation is not known as definitely as for some of the other pans. Although this factor has been determined for the Colorado pan (Type (3)) under a wide range of conditions, this type has been used to only a limited extent in evaporation investigations, and, consequently, comparative evaporation records are available for only a few places. As previously mentioned, records observed from floating pans are unreliable, and since the evaporation from a sunken land pan is almost identical with that from a floating pan of the same size, there is no purpose in using a floating pan.

PROCEDURE

The location of the evaporation pan should be given careful consideration; otherwise, comparable and reliable data can not be obtained. The land pans should be installed in level areas unobstructed by trees and buildings, and if the observations are for the purpose of determining the equivalent reservoir evaporation, the location chosen should be representative of conditions at the reservoir. Isolated places, where no satisfactory water supply is available, or where it is not possible to secure the services of an intelligent observer, should be avoided. The equipment should be protected by a close mesh-wire fence. Fig. 5 is a diagram of a completely equipped Class *A* Weather Bureau Station. Although floating pans are not recommended, if it is necessary to install one for any reason, the location chosen should be in an area protected, to some extent, from the full force of the waves. The exposure to the wind, however, should represent the average conditions for the reservoir. The approximate elevation of the area chosen should be known, because evaporation varies with altitude.

The evaporation from the different types of pans is dependent, to some extent, on the color of the pan, and for this reason it is recommended that no paint, tar, or other coating, be used. To reduce the growth of algæ and other plants in the water, the pans should be cleaned at least once a month, and oftener if necessary. Cleaning the tanks also reduces the concentration of salts in the water, which otherwise would occur due to the evaporation.

In order to obtain comparable evaporation data, complete meteorological records should be taken and a standardized procedure should be followed in taking the observations. The meteorological record should include the air and water temperature, the humidity, the wind movement, the precipitation, and the evaporation loss. The following instruments are required:

Two sets of "maximum-and-minimum" thermometers for determining the mean air and water temperature; or
Two ordinary thermometers where observations can be taken at 12-hr intervals, or oftener;
One psychrometer for determining the humidity of the air;
One anemometer for recording the wind movement;
One rain gauge for measuring the precipitation;
One gauge for measuring the evaporation; and
One instrument shelter for the air thermometers and the psychrometer.

The standard Weather Bureau practice should be followed in taking the observations. Complete instructions for the operation of Pan (1) are given in *Circular L* of the Instrument Division of the U. S. Weather Bureau, and observations on the other types of pans should follow the same procedure. The mean water temperature and the relative humidity of the air are not observed by the Weather Bureau at its Class *A* stations, but these observations should be taken. The mean temperature of the water may be determined either by readings of "maximum-and-minimum" thermometers floating in the water with their bulbs immersed $\frac{1}{4}$ in. beneath the surface of the water, or by readings on an ordinary thermometer similarly exposed. The "maximum-and-minimum" thermometers require only a single reading daily to obtain a fair average of the temperature, whereas the ordinary thermometer must be read twice daily at 12-hr intervals for similar accuracy. Maximum thermometers are easily broken, however, and for this reason are not as satisfactory when the readings are taken by amateur observers. Where possible, the sling or rotating psychrometer should be used for determining the humidity, but where the readings can not be taken at regular intervals, a hair hygrometer may be used to obtain approximate results.

The velocity of the wind varies with the elevation above the ground surface. For this reason, a standard anemometer setting should be adopted for each type of evaporation pan in order to obtain comparable results. The standard practice at Class *A* Weather Bureau stations is to mount the anemometers on the grillage of timbers which supports the pan so that the cups of the anemometer are 6 in. above the top of the pan (see Fig. 3). At stations of the Bureau of Plant Industry, the anemometer is placed so that the cups are 24 in. above the top of the pan. Although no standard has been adopted for installing the anemometers at stations where Pans (3) or (4) are used, it is suggested for uniformity that the anemometers be mounted so that their cups are 18 in. above the surface of the ground or the water. Either 3-cup or 4-cup anemometers, of the Weather Bureau pattern, may be used. The 3-cup anemometers are more accurate at high wind velocities, but near the ground the velocities are never so high that the 4-cup anemometer is not sufficiently accurate.

The rain gauge for evaporation stations should be of the Weather Bureau type and should be installed in accordance with the instructions in *Circular L* of the Weather Bureau. The evaporation gauges used in connection with the different pans are usually of special forms, but any gauge giving readings to within 0.001 ft should be satisfactory.

Observations should be taken at 7:00 A. M., according to the recommendations of the Weather Bureau, but where complete and accurate meteorological data require it, the readings should be taken oftener. Observations twice daily at 12-hr intervals (usually at 7:00 A. M. and 7:00 P. M.) give more accurate results, particularly as to the humidity. Where readings can not be taken at regular intervals, recording instruments should be used to obtain the records.

CONCLUSIONS

Records from floating pans are not as consistent or reliable as land-pan records; nor is the evaporation from a floating pan any nearer the evaporation from a large water surface than that from a sunken pan of the same size and shape.

Comparisons between the evaporation from Class *A* land pans of the Weather Bureau (Type (1)) and Colorado sunken pans (Type (3)) with the evaporation from large water surfaces, indicate that there is a definite relation between the pan and reservoir evaporation. For the Class *A* land pan, the factor for computing the reservoir evaporation from the pan evaporation is between 0.69 and 0.70, and for the Colorado pan, it is 0.78.

Comparison of the evaporation from different types of pans with that from large water surfaces of different sizes, shows that the size of the pan has a proportionately smaller effect on the evaporation as the size of the surface increases, and that when the diameter is greater than 12 ft, the size of the pan has practically no effect on the evaporation.

When all factors are considered, Pan (1) is probably better suited for evaporation investigations than the other evaporation pans. In order to obtain comparable evaporation results, standard equipment installed under representative conditions should be used, and a standard procedure should be followed in making the observations.

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The author wishes to acknowledge the co-operation of those agencies and individuals who furnished evaporation data from their unpublished records, or who gave other information about the experiments.

APPENDIX

Summary of Data on Evaporation from Water Surfaces

For convenience of reference in this Appendix the various pans used in evaporation studies are identified by numbers as defined in the paper. For example, Pan (1) is the Class *A* land pan of the U. S. Weather Bureau, shown in Figs. 1 and 5; Pan (2) is the sunken pan of the U. S. Bureau of Plant Industry, shown in Fig. 6; Pan (3) is the Colorado sunken pan shown in Figs. 1, 3, and 7; Pan (4) is the floating pan of the U. S. Geological Survey shown in Fig. 8; and Pan (5) is simply Pan (1) used as a floating pan, as shown in Fig. 4.

East Park Reservoir, Stonyford, Calif.—The data in Table 1 are from records of the Bureau of Agricultural Engineering, United States Department of Agriculture, for observations made in July, 1930.[8] Water was maintained at a level approximately 2 in. below the rim in Pans (1) and (3). In Pans (2) and (4), the level was 3 in. below the rim.

TABLE 1.—Evaporation from Reservoir Compared with That from Pans; East Park Reservoir, Stonyford, Calif.

Item No.	Water surface	Dimensions		Precipitation, in inches	Temperature, in Degrees, Fahrenheit		Difference in vapor pressure, in inches	Velocity of wind, in miles per hour	Evaporation; total for the month, in inches	Ratio of evaporation, Item No. 1 to other items
		Area	Depth, in feet		Air	Water				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	East Park Reservoir	1800 acres	....	0.0	82.12	80.72	0.6723	1.676	8.183	1.00
2	Pan (3)	3 ft. square	1.5	0.0	78.42	79.32	0.6395	1.940	9.498*	0.75
3	Pan (2)	Diameter, 4 ft.	3.0	0.0	78.42	79.77	0.6393	1.905	10.464	0.78
4	Pan (1)	Diameter, 4 ft.	0.83	0.0	78.42	73.83	0.4998	2.667	11.926	0.69
5	Pan (5)	Diameter, 4 ft.	0.83	0.0	78.53	77.91	0.5680	1.779	10.432	0.78

* Data for the period beginning July 19, 1930. The corresponding evaporation from the reservoir for the same period is 7.162.

Pans (2) and (3) were buried to within 3 in. and 1 in. of the top, respectively. Pan (1) was installed on a grillage of timbers, and Pan (5) was fixed in a raft. In Column (9), Table 1, all the anemometer readings have been reduced to surface velocities. East Park Reservoir is in a water-tight valley, geologically. Its elevation is 1 200 ft.

Comparison of Evaporation from Pan (1) and Various Other Types of Pans.—The data in Table 2 are largely self-explanatory. In Table 2(*a*) a comparison is afforded between evaporation from Pans (1) and (2) (see Columns (8) and (9)), a Class *A* land pan, and a sunken pan. This part of Table 2 was compiled from information presented[9] by Ivan E. Houk, M. Am. Soc. C. E., covering observations made during 1926, 1927, and 1928. Pan (1) was the standard type shown in Fig. 5, while Pan (2) was 4 ft in diameter

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[8] *Technical Bulletin No. 271*, U. S. Dept. of Agriculture.

[9] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), Table 9, p. 991.

TABLE 2.—COMPARISON BETWEEN EVAPORATION FROM PAN (1), AND VARIOUS OTHER TYPES OF PANS

Item No. (1)	Month (2)	Number of years of record (3)	Precipitation, in inches (4)	Temperature of air, in degrees Fahrenheit (5)	Relative humidity (percentage) (6)	Velocity of wind, in miles per hour (7)	EVAPORATION: TOTAL FOR MONTH, IN INCHES Pan (1) (8)	(9)	Ratio: Column (9) to Column (8) (10)
(a) LOS GRIEGOS, NEW MEXICO, COLUMN (9), IS TYPE (2); 4 FEET IN DIAMETER AND 2 FEET DEEP									
	Pans						Type (1)	Type (2)	
1	Jan......	2	0.0	36.2	48	2.3	1.74	1.04	0.60
2	Feb......	2	0.33	41.1	50	3.6	3.06	2.12	0.69
3	March...	2	0.28	46.8	40	4.2	6.20	4.46	0.72
4	April.....	2	0.46	52.9	37	4.6	8.50	5.86	0.69
5	May.....	2	0.69	61.6	34	4.0	10.68	7.28	0.68
6	June.....	2	0.50	68.2	34	3.4	11.46	7.74	0.68
7	July.....	2	1.12	74.1	49	2.6	11.12	7.60	0.68
8	Aug.....	2	2.14	70.8	61	2.6	8.76	7.02	0.80
9	Sept.....	3	0.84	65.7	55	2.6	7.02	5.26	0.75
10	Oct......	2	0.61	56.0	56	2.3	5.27	3.82	0.72
11	Nov.....	2	0.02	45.4	52	3.0	3.70	2.72	0.74
12	Dec......	2	0.54	33.4	72	2.8	1.36	0.98	0.72
13	Totals...	......	7.53	....	....	....	78.87	55.90	....
14	Means...	......	....	54.3	49	3.2	....	....	0.71
(b) GARNETT, COLORADO, COLUMN (9) IS TYPE (2); 4 FEET IN DIAMETER AND 2 FEET DEEP									
	Pans						Type (1)	Type (2)	
15	April.....	1	0.36	41.9*	....	4.2	....	....	0.79
16	May.....	2	1.14	48.4*	....	3.7	....	....	0.76
17	June.....	3	1.00	57.4*	....	3.4	....	....	0.78
18	July.....	3	2.16	61.6*	....	3.1	....	....	0.77
19	Aug.....	3	1.44	59.5*	....	1.8	....	....	0.76
20	Sept.....	3	1.00	53.5*	....	2.0	....	....	0.81
21	Oct......	3	0.45	43.8*	....	1.9	....	....	0.80
22	Totals...	......	7.55	....	....	....	....	....	....
23	Means...	......	....	52.3*	....	2.9	....	....	0.78
(c) LINCOLN, NEBRASKA, COLUMN (9) IS BRIGGS PAN, TYPE (2); 7.5 FEET IN DIAMETER AND 2 FEET DEEP									
	Pans						Type (1)	Briggs Pan	
24	May.....	4	3.45	61	66	4.8	6.603	5.181†	0.78
25	June.....	4	4.38	72	65	3.6	8.789	6.386	0.73
26	July.....	4	1.73	78	60	3.0	10.250	7.094	0.69
27	Aug.....	4	3.14	74	65	2.7	8.592	5.921	0.69
28	Sept.....	4	2.61	66	66	3.5	6.612	4.612†	0.70
29	Oct......	4	2.51	54	67	4.1	4.588	3.160	0.69
30	Totals...	......	17.82	....	....	....	45.434	32.354	....
31	Means...	......	....	68	65	3.6	....	....	0.71
(d) MILFORD, UTAH, COLUMN (9) IS TYPE (2); 12 FEET IN DIAMETER AND 3 FEET DEEP									
	Pans						Type (1)	Type (2)	
32	March...	1	0.90	41.3	54	4.4‡	3.98§	2.74§	0.69
33	April.....	2	0.64	49.3	53	4.3	8.42‖	5.54‖	0.66
34	May.....	2	0.82	57.7	39	4.9	11.19	7.74	0.69
35	June.....	2	0.02	65.2	38	4.8	14.71	9.92	0.67
36	July.....	2	0.66	73.3	39	4.2	14.02	9.40	0.67
37	Aug.....	3	0.23	70.0	45	4.6	13.39¶	9.02¶	0.67
38	Sept.....	3	0.21	60.0	41	4.9	10.75	7.23	0.67
39	Oct......	3	1.12	50.1	44	3.7	6.59	4.40	0.67
40	Totals...	......	4.60	....	....	....	83.05	55.99	....
41	Means...	......	....	58.4	44	4.5	....	....	0.67

* Temperature of water. † Three-year record. ‡ From *Water Supply Paper 617*, U. S. Geological Survey, p. 36. § Twelve days, March, 1926, estimated. ‖ Twenty-three days, April, 1927, estimated. ¶ Four days, August, 1925, estimated.

TABLE 2.—(*Continued*)

Item No. (1)	Month (2)	Number of years of record (3)	Precipitation, in inches (4)	Temperature of air, in degrees Fahrenheit (5)	Relative humidity (percentage) (6)	Velocity of wind, in miles per hour (7)	EVAPORATION: TOTAL FOR MONTH, IN INCHES Pan (1) (8)	(9)	Ratio: Column (9) to Column (8) (10)
(*e*) NELSON RESERVOIR, MONTANA, COLUMN (9) IS TYPE (5); 4 FEET IN DIAMETER AND 10 INCHES DEEP									
	Pans						Type (1)	Type (5)	
42	May	3	2.37	54.5	64	6.64	8.82	5.26	0.60
43	June	3	5.06	65.9	64	5.80	9.57	6.37	0.67
44	July	3	2.99	69.6	63	4.65	11.18	7.82	0.70
45	Aug.	3	0.95	64.2	59	4.69	11.36	8.67	0.76
46	Sept.	3	0.76	58.7	60	5.01	7.70	5.31	0.69
47	Oct.	3	0.09	47.9	66	5.32	4.26	2.78	0.65
48	Totals...	......	12.22	....	....	....	52.89	36.21	....
49	Means...	......	....	60.1	63	5.35	....	....	0.68
(*f*) OAKDALE, CALIFORNIA, COLUMN (9) IS TYPE (5); 4 FEET IN DIAMETER AND 10 INCHES DEEP									
	Pans						Type (1)	Type (5)	
50	June	1	0.00**	70.8	....	5.9	13.373	8.233	0.62
51	July	1	0.00	78.5	....	5.8	17.034	10.849	0.64
52	Aug.	1	0.00	73.6	....	5.5	14.651	9.303	0.63
53	Sept.	1	0.06	68.8	....	4.6	10.351	6.839	0.66
54	Totals...	......	0.06	....	....	....	55.409	35.224	....
55	Means...	......	....	72.9	....	5.4	....	....	0.64
(*g*) FALL RIVER MILLS, CALIFORNIA, COLUMN (9) IS TYPE (5); 4 FEET IN DIAMETER AND 8 INCHES DEEP									
	Pans						Type (1)	Type (5)	
56	Jan.	5	2.30	33.1	....	1.5	....	1.085	....
57	Feb.	5	2.79	38.6	....	2.0	2.034††	1.755	0.86
58	March	5	2.23	44.4	....	2.5	3.635‡‡	3.199	0.88
59	April	5	1.73	48.2	....	3.1	4.903	4.255	0.87
60	May	5	1.13	55.4	....	2.9	7.670	6.356	0.83
61	June	5	0.59	63.4	....	2.8	10.209	7.768	0.76
62	July	5	0.00**	69.3	....	2.8	13.032	9.226	0.71
63	Aug.	5	0.06	66.1	....	2.8	11.445	7.732	0.68
64	Sept.	5	0.57	57.5	....	2.4	7.235	5.349	0.74
65	Oct.	5	0.85	50.8	....	1.8	4.581	3.316	0.72
66	Nov.	5	3.39	42.1	....	1.6	....	1.659	....
67	Dec.	5	1.72	34.1	....	1.5	....	1.368	....
68	Totals...	......	17.36	....	....	....	64.744	48.956	....
69	Means...	......	....	50.2	....	2.3	....	....	0.76

** Trace. †† Two-year record. ‡‡ Four-year record.

and 2 ft deep, with 3 in. of the tank above ground. Water in Pan (2) was maintained at ground level. The anemometer cups were fixed 24 in. above ground, and readings were taken generally at 5:00 P. M. Los Griegos, N. Mex., is at an elevation of 4 970 ft.

Table 2(*b*) is from unpublished data in the office of the State Engineer of Colorado, made available through the courtesy of R. J. Tipton, Assoc. M. Am. Soc. C. E. Observations were made in 1927, 1928, and 1930. Pans (1) and (2) were the same dimensions as given in Table 2(*a*), with the difference that Pan (2) was buried to within 2 in. above ground. Mariotte control apparatus[10] was used to maintain the level in the pans. A 3-cup anemometer was fixed at 18 in. above the ground. Garnett, Colo., is at Elevation 7 700.

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[10] For description, see *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 962.

Table 2(*c*)[11] is from data observed during the four years, 1917 to 1920, inclusive. The Briggs pan (Type (2)) was 7:5 ft in diameter and 2.0 ft deep, and was set with the rim flush with the ground. Water was maintained 4 in. below the rim in both pans, and the anemometer cups were 5 in. above Pan (1). The relative humidity readings in Column (6) were taken at Lincoln, Nebr. (3 miles from the pans) at 7: 00 A. M. and 7: 00 P. M. The pans, in this case, were at Elevation 1 150.

Table 2(*d*)[12] is from data observed during the three years, 1925, 1926, and 1927. Pan (2) was 12 ft in diameter, was set with the rim 3 in. above ground, and the water was maintained at ground level. The relative humidity in Column (6), Table 2(*d*), was taken from the record of Modena, Utah. Evaporation readings were taken at 8: 00 A. M. The elevation of Milford, Utah, is 4 962.

Table 2(*e*) shows[13] some data and observations made during the three years, 1921, 1922, and 1923. Water in the floating pan, Type (5), was 6 in. deep, and the surface was maintained at the same elevation as the water in the reservoir. This pan was 350 ft from shore; the land pan, Type (5), was on a small knoll, 100 yd from shore. Anemometer readings were begun in 1922, with standard setting. Precipitation records (Column (4)) are those of Malta, Mont., and the relative humidity (Column (6)) readings are those at Havre, Mont. Nelson Reservoir is at Elevation 2 215.

The data pertaining to the floating pans in Tables 2(*f*) and 2(*g*) are from unpublished records furnished by the courtesy of the U. S. Weather Bureau. The land-pan records are from Climatological Data, California Section, U. S. Weather Bureau. The data in Table 2(*f*) were recorded during 1921. The land pan (Type (1)) was placed on a knoll and was filled with water to about 7 in., the anemometer cups being 6 in. above the rim. Observations were taken daily at 7: 00 A. M. The floating pan (Type (5)) was exposed n a raft and filled to a depth of 6 to 7 in. of water. Oakdale, Calif., is at Elevation 215.

Data in Table 2(*g*) were recorded during the five years, 1925 to 1930. The land pan (Type (1)) was filled with water to a depth of about 7 in., anemometer cups being 11 in. above the rim. The floating pan (Type (5)) was submerged 4 in. and was filled with water to a depth of 6 in. Observations were taken daily at 7: 30 A. M. Fall River Mills, Calif., is at Elevation 3 340, and the ground surrounding the station is flat.

Comparison Between Evaporation from Different Sizes of Pans.—Data compiled in Table 3(*a*) are from records of the U. S. Bureau of Agricultural Engineering, covering the years 1926, 1927, and 1928.[8] The wind velocity given is at the surface of the ground or water in each case. The wind velocities for Pans (3) and (4) are the same as for the reservoir. The elevation of Fort Collins, Colo., is 5 000 ft.

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[11] Comp. from *Monthly Weather Review,* Vol. 48, December, 1920, p. 715.

[12] Comp. from Climatological Data, Utah Section, U. S. Weather Bureau.

[13] Comp. from Climatological Data, Montana Section, and from the Annual Rept. of the Chief of the U. S. Weather Bureau; see *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 275.

TABLE 3.—COMPARISON BETWEEN EVAPORATION FROM DIFFERENT SIZES OF PANS

Item No.	Month	Number of years of record	PAN A					PAN B				PAN C				PAN D					Precipitation, in inches
			TEMPERATURE, IN DEGREES FAHRENHEIT		Difference in vapor pressure, in inches	Wind velocity, in miles per hour	Total evaporation for the month, in inches	Temperature of water, in degrees Fahrenheit	Difference in vapor pressure, in inches	Total evaporation for the month, in inches	Ratio: Column (8) to Column (11)	Temperature of water, in degrees Fahrenheit	Difference in vapor pressure, in inches	Total evaporation for the month, in inches	Ratio: Column (8) to Column (15)	Temperature of water, in degrees Fahrenheit	Difference in vapor pressure, in inches	Wind velocity, in miles per hour	Total evaporation for the month, in inches	Ratio: Column (8) to Column (20)	
			Air	Water																	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
	(a) FORT COLLINS, COLORADO																				
1	Pans.......		Reservoir: 85 Feet in Diameter and 6.75 Feet Deep					Pan (4): 3 Feet Square and 18 Inches Deep				Pan (3): 3 Feet Square and 18 Inches Deep				Pan (1): 4 Feet in Diameter and 10 Inches Deep					
2	April.......	2	45.6	50.8	0.1691	2.67	3.14	50.1	0.1677	4.08	0.77	49.6	0.1846	4.18	0.75	48.4	0.1857	3.71	5 21	0.60	1.408
3	May.......	2	58.3	62.5	0.2526	2.11	4.40	62.1	0.2530	5.80	0.76	61.6	0.2746	5.78	0.76	60.4	0.2752	2.91	6.94	0.63	2.176
4	June.......	2	62.3	68.5	0.2812	1.16	3.75	67.9	0.2800	4.94	0.76	66.6	0.2848	4.85	0.77	65.2	0.2699	1.70	5.42	0.69	2.565
5	July.......	2	70.5	75.8	0.3776	1.04	4.99	75.0	0.3709	6.59	0.76	74.1	0.3956	6.54	0.76	73.0	0.3842	1.58	7.21	0.69	1.229
6	Aug.......	2	67.2	72.3	0.3372	1.03	4.47	71.7	0.3313	5.82	0.77	70.3	0.3450	5.96	0.75	68.7	0.3203	1.56	6.26	0.71	1.128
7	Sept.......	3	57.4	64.7	0.3063	1.17	4.19	63.9	0.3014	5.22	0.80	60.9	0.2891	4.90	0.86	58.6	0.2607	1.76	5.10	0.82	0.522
8	Oct.......	3	48.8	54.4	0.1998	1.34	2.71	53.7	0.1974	3.59	0.76	51.4	0.1938	3.30	0.82	50.1	0.1876	2.05	3.77	0.72	0.792
9	Nov.......	3	37.9	41.1	0.0974	1.68	1.42	40.0	0.0898	1.74	0.82	37.1	0.0747	1.44	0.99	37.2	0.0782	2.60	1.84	0.77	0.474
10	Totals.....	......	......	......	......	......	29.07	......	......	37.78	......	......	......	36.95	......	......	......	......	41.75	......	10.291
11	Means.....	....	56.0	61.3	0.2526	1.52	......	60.6	0.2489	......	0.77	59.0	0.2553	......	0.79	57.7	0.2452	2.23	......	0.70	......
	(b) SALT CREEK BRIDGE, CALIFORNIA (CIRCULAR, FLOATING, TYPE (5) PAN)																				
12	Pans.......		Pan (5): 12 Feet in Diameter and 10 Inches Deep					Pan (5): 6 Feet in Diameter and 10 Inches Deep				Pan (5): 4 Feet in Diameter and 10 Inches Deep				Pan (5): 2 Feet in Diameter and 10 Inches Deep					
13	May.......	5*	88.2	82.8	0.646	4.2	12.12	82.4	0.631	12.80	0.95	82.6	0.640	14.79	0.82	82.9	0.648	......	14.54	0.83	......
14	June.......	30*	86.0	82.6	0.628	5.0	10.89	82.0	0.604	11.61	0.94	82.0	0.609	13.29	0.82	81.9	0.601	......	13.83	0.79	......
15	July.......	6*	87.8	84.1	0.646	4.5	11.90	83.3	0.618	12.37	0.96	83.5	0.622	14.26	0.84	83.3	0.616	......	14.54	0.82	......
16	Totals......	......	......	......	......	......	34.91	......	......	36 78	......	......	......	42.34	......	......	......	......	42.91	......	......
17	Means.....	......	86 53	82.84	0.633	4.83	......	82.24	0.609	......	0.95	82.29	0.615	......	0.82	82.23	0.609	......	......	0.81	......

* Days.

(*c*) Salton Sea, California (Circular Land Pan, Type (1))

18	Pans.......		Pan (1): 6 Feet in Diameter and 10 Inches Deep									Pan (1): 4 Feet in Diameter and 10 Inches Deep				Pan (1): 2 Feet in Diameter and 10 Inches Deep					
19	April.......	18*	75.9	75.5	0.589	4.93	11.58	......	......	......	......	76.1	0.616	11.52	1.01	76.1	0.610	......	13.44	0.86	......
20	May.......	31*	79.9	79.2	0.572	7.08	13.02	......	......	......	......	79.5	0.587	13.95	0.93	79.3	0.576	......	16.86	0.77	......
21	June.......	30*	83.8	83.7	0.710	7.03	13.62	......	......	......	......	83.3	0.694	15.48	0.88	82.9	0.681	......	18.81	0.72	......
22	July.......	30*	88.5	88.9	0.690	6.67	13.70	......	......	......	......	88.9	0.675	15.81	0.87	88.9	0.661	......	19.62	0.70	......
23	Totals.....	....	......	......	......	......	51.92	......	......	......	......	......	......	56.76	......	......	......	......	68.73	......	......
24	Means.....	....	82.68	82.50	0.6453	6.72	......	......	......	......	......	82.57	0.6455	......	0.92	82.40	0.6339	......	......	0.76	......

(*d*) Brawley, California (Circular Land Pan, Type (1))

25	Pans.......		Pan (1): 6 Feet in Diameter and 10 Inches Deep									Pan (1): 2 Feet in Diameter and 10 Inches Deep									
26	March 5†...	4*	70.7	70.2	0.504	0.9	0.193‡	......	......	......	......	......	......	......	......	71.6	0.539	......	0.283‡	0.68	......
27	March 11†..	4*	68.4	67.3	0.476	1.0	0.228‡	......	......	......	......	......	......	......	......	67.6	0.488	......	0.295‡	0.77	......
28	March 17†..	3*	71.2	66.2	0.374	2.1	0.240‡	......	......	......	......	......	......	......	......	71.2	0.496	......	0.315‡	0.76	......
29	March 26†..	5*	64.4	65.1	0.362	4.5	0.283‡	......	......	......	......	......	......	......	......	66.0	0.382	......	0.346‡	0.82	......
30	April 2†....	5*	66.0	65.1	0.453	2.4	0.264‡	......	......	......	......	......	......	......	......	65.5	0.461	......	0.362‡	0.73	......
31	April 16‡...	5*	61.9	65.5	0.390	3.1	0.276‡	......	......	......	......	......	......	......	......	66.6	0.413	......	0.354‡	0.78	......
32	April 23‡...	5*	79.9	75.7	0.701	1.7	0.350‡	......	......	......	......	......	......	......	......	77.4	0.748	......	0.453‡	0.77	......
33	April 30‡...	5*	80.8	75.9	0.654	3.0	0.409‡	......	......	......	......	......	......	......	......	77.2	0.693	......	0.532‡	0.77	......
34	Means.....	....	70.4	68.9	0.489	2.3	0.280‡	......	......	......	......	......	......	......	......	70.4	0.528	......	0.368‡	0.76	......

(*e*) Mecca, California (Circular Land Pan, Type (1))

35	Pans.......		Pan (1): 6 Feet in Diameter and 10 Inches Deep									Pan (1): 2 Feet in Diameter and 10 Inches Deep									
36	March 5‡...	5	68.4	65.8	0.417	2.4	0.272‡	......	......	......	......	......	......	......	......	66.4	0.429	......	0.362‡	0.75	......
37	March 12‡..	5	67.5	64.0	0.413	2.3	0.244‡	......	......	......	......	......	......	......	......	64.4	0.421	......	0.335‡	0.73	......
38	March 17‡..	3	69.8	68.7	0.425	3.3	0.264‡	......	......	......	......	......	......	......	......	69.4	0.441	......	0.398‡	0.66	......
39	March 25‡..	4	64.4	65.3	0.362	3.7	0.276‡	......	......	......	......	......	......	......	......	64.8	0.350	......	0.378‡	0.73	......
40	April 2‡....	5	66.4	65.8	0.468	2.9	0.311‡	......	......	......	......	......	......	......	......	64.9	0.449	......	0.449‡	0.69	......
41	April 9‡....	3	74.8	71.2	0.465	4.0	0.346‡	......	......	......	......	......	......	......	......	72.3	0.492	......	0.496‡	0.70	......
42	April 16‡...	5	67.1	66.9	0.394	3.1	0.283‡	......	......	......	......	......	......	......	......	66.9	0.394	......	0.382‡	0.74	......
43	April 23‡...	5	82.6	73.2	0.669	3.2	0.449‡	......	......	......	......	......	......	......	......	73.8	0.685	......	0.642‡	0.70	......
44	April 29‡...	4	81.7	75.6	0.622	2.7	0.417‡	......	......	......	......	......	......	......	......	76.3	0.646	......	0.559‡	0.75	......
45	Means.....	....	71.4	68.5	0.471	3.1	0.318	......	......	......	......	......	......	......	......	68.8	0.479	......	0.445‡	0.71	......

* Days. † Period ending. ‡ Per 24 hours.

Tables 3(*b*) and 3(*c*) were compiled from unpublished data for 1910 furnished by the courtesy of the U. S. Weather Bureau. The pans in Table 3(*b*) were floated on a raft in an arm of the Salton Sea (approximate Elevation − 210), and were kept well immersed. The anemometer cups were approximately 8 in. higher than the rims of the pans.

The land pans in Table 3(*c*) were set on a platform 8 or 10 ft above the Salton Sea (approximate Elevation − 200), about ½ mile from the shore. Water was maintained in them at a depth of approximately 7 in. The anemometer cups were approximately 8 in. higher than the rims of the pans.

TABLE 4.—COMPARISON BETWEEN EVAPORATION FROM A 12-FOOT CIRCULAR SUNKEN PAN (TYPE (2)) AND VARIOUS OTHER TYPES

Item No. (1)	Type of pan (2)	Diameter, in feet (3)	Depth, in feet (4)	Depth of water in pan, in feet (5)	Depth of setting, in feet (6)	PERIOD OF OBSERVATION				EVAPORATION FOR THE PERIOD, IN INCHES		Ratio: Col. (11) to Col. (12) (13)
						From (7)	To (8)	Except (9)	Days of record (10)	Item No. 1 (11)	Given pan (12)	
1	2	12.0	3.0	2.75	2.75	11/15/15	9/30/17	....	684	85.47	85.47	1.00
2	2	1.0	3.0	2.75	2.75	11/15/15	9/30/17	12/ 4/16 to 5/ 7/17	428	79.68	126.59	0.63
3	2	2.0	3.0	2.75	2.75	11/15/15	11/13/16	....	363	49.16	63.12	0.78
4	2	3.39	3.0	2.75	2.75	11/15/15	9/17/17	11/21/16 to 3/11/17	183	80.51	96.74	0.83
5	2	6.0	3.0	2.75	2.75	11/15/15	9/30/17	....	684	85.47	92.87	0.92
6	2	9.0	3.0	2.75	2.75	11/15/15	11/13/16	....	363	49.16	49.63	0.99
7	3	2.0†	3.0	2.75	2.75	4/17/16	9/24/17	11/13/16 to 5/29/17	328	68.18	89.49	0.76
8	3	3.0†	3.0	2.75	2.75	11/15/15	9/24/17	11/13/16 to 5/ 8/17	502	78.33	98.63	0.79
9	1	4.0	0.83	0.62	0.00‡	11/15/15	9/30/17	12/13/15 to 3/ 6/16 and 11/13/16 to 4/11/17	452	81 69	116.09	0.70
10	1	2.0	0.83	0.62	0.00‡	4/11/17	9/30/17	....	172	32.53	54.91	0.59
11	1	6.0	0.83	0.62	0.00‡	4/11/17	7/30/17	....	110	20.05	26.66	0.75
12	2	2.0	0.5	0.25	0.25	6/ 5/16	7/30/17	10/ 9/16 to 5/28/17	189	43.47	55.62	0.78
13	2	2.0	1.0	0.75	0.75	6/ 5/16	9/24/17	10/ 9/16 to 5/28/17	244	55.24	70.16	0.79
14	2	2.0	1.5	1.25	1.25	6/ 5/16	9/24/17	10/ 9/16 to 5/28/17	215	55.24	70.69	0.78
15	2	2.0	2.0	1.75	1.75	6/ 5/16	7/30/17	10/ 9/16 to 5/28/17	189	43.47	55.30	0.79
16	2	2.0	6.0	5.75	5.75	6/ 5/16	9/24/17	10/ 9/16 to 5/28/17	215	55.24	70.39	0.78
17	2	6.0	1.0	0.75	0.75	6/ 5/16	10/ 9/16	....	97	28.49	30.48	0.93
18	2	6.0	2.0	1.75	1.75	6/ 5/16	10/ 9/16	....	97	28.49	30.36	0.94
19	4	3.0†	1.5	1.25	1.25§	4/17/16	9/17/17	*	....	64.06	72.04	0.89
20	5	0.83	0.83	0.75	0.75	7/31/17	9/17/17	....	72	10.27	13.42	0.77
21	5	2.0	1.0	0.75	0.75	5/10/17	9/17/17	*	....	14.29	16.04	0.89
22	5	4.0	1.0	0.75	0.75	7/31/17	9/17/17	....	48	10.27	10.63	0.97
23	5	6.0	1.0	0.75	0.75	5/10/17	8/27/17	*	....	6.57	6.39	1.03

* Intermittent periods. † Square. ‡ Above ground. § Depth not given, but assumed to be standard.

Tables 3(*d*) and 3(*e*) are from unpublished records of 1910 furnished by the courtesy of the U. S. Weather Bureau. Water in the pans was maintained at an approximate depth of 8 in. In Table 3(*d*) anemometer cups were approximately 7 in., and in Table 3(*e*), 8.8 in., higher than the rim. The water surface was fully exposed. Brawley, Calif., is at Elevation − 110, and Mecca, Calif., is at Elevation − 189.

Comparison Between Evaporation from 12-Foot Circular Sunken Pan (Type (2)) and Various Other Types.—All the data in Table 4 were observed at Denver, Colo. (Elevation 5 346), during the years 1915, 1916, and 1917.[14]

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[14] Comp. from *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917, and from *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Table 15, p. 308.

General Comparisons: Land Pans and Floating Pans.—Table 5(*a*) has been compiled from three sources: *Bulletin 9* of the Department of Engineering, State of California; Climatological Data, California Section, U. S. Weather Bureau; and, *Transactions,* Am. Soc. C. E., Vol. 90 (June, 1927). Buena Vista Lake has an area of about 10 000 acres and is at Elevation 420. The observations were made in 1920, the temperature and precipitation records being taken at Bakersfield, Calif. The land pan—Type (3)—was sunk to within 3 in. of the top; the floating pan—Type (4)—was always submerged to within 6 in., of the top, or less. The depth of water was maintained at 3 in. below the top of both pans.

Table 5(*b*) is compiled from records in the *Water Supply Papers* of the U. S. Geological Survey, collected between 1916 and 1925. The Class *A* land pan (Type (1)) and the floating pan of the U. S. Geological Survey (Type (4)) were both of standard dimensions, as shown in Figs. 5 and 8, respectively. The floating pan was set in a lake 20 by 150 ft and from 8 to 10 ft lower than the land pan. Observations were taken at 7:00 A. M. Austin, Tex., is at Elevation 475.

Tables 5(*c*), 5(*d*), and 5(*e*) are from the set-up of identical pans at Morena Reservoir, Barrett Reservoir, and Lower Otay Reservoir, California, during 1928. The precipitation for all three was taken at the Barrett Dam and the temperature at Escondido, Calif. Some of the records were furnished by the courtesy of H. N. Savage, M. Am. Soc. C. E., and others were taken from Climatological Data, California Section, U. S. Weather Bureau. The elevations of the stations are as follows: Morena, 3 000 ft; Barrett, 1 600 ft; anod Lower Otay, 500 ft, above sea level.

Table 5(*f*) was compiled from data presented[15] by Charles H. Lee, M. Am. Soc. C. E., and from the Annual Reports of the Chief of the Weather Bureau. The floating pan (Type (4)), 3 ft square and 10 in. deep, was placed on a raft in Owens River. Water was kept approximately 2 in. below the rim. The deep land pan (Type (2)) was set with the top flush with the ground. In this pan water was never more than 4 in. below the rim. The shallow land pan (Type (3)) was set with the top 5 in. above ground and the water level was maintained approximately 2 in. below the rim. The anemometer cups were 42 ft above ground. Some of the evaporation records were partly estimated. Independence, Calif., is at Elevation 3 907; the period of record was from 1908 to 1911.

Table 5(*g*) was compiled from published and unpublished data recorded[16] by Mr. C. M. O'Neil, of the Canadian Pacific Railroad Company, during 1923 and 1924. The dimensions of land and floating pans were alike. The land pans were sunk in the ground to within 2 in. of their tops. Water was kept at ground level. In the floating pans, water was kept 4 in. below the rim or slightly above the level of submergence. The anemometer cups were 2 ft above ground. Temperature and wind velocity data were taken from plotted data. Brooks Reservoir, Alberta, Canada (Elevation 2 450), has an area of about 40 acres.

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[15] *Transactions,* Am. Soc. C. E., Vol. LXXVIII (1915), pp. 177–178.

[16] *Loc. cit.,* Vol. 90 (June, 1927), pp. 343–351.

TABLE 5.—General Comparison of Evaporation Data from Land Pans and Floating Pans

Item No.	Month	Number of years of record	Precipitation, in Inches		Temperature, in Degrees Fahrenheit				Total Evaporation for the Month, in Inches			Ratio of Evaporation			Relative humidity (percentages)	Wind velocity, in miles per hour
					Air	Water						Column (11) to Column (10)	Column (12) to Column (11)	Column (12) to Column (10)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
(*a*) Bakersfield, California (Buena Vista Lake); Pans (3) and (4), 3 Feet Square and 2 Feet Deep																
1	Pan								Pan (3)	Pan (4)	Lake					
2	Jan.	1	0.51		47.6	......	......	......	1.82	1.82	......	1.00	......	......	......	......
3	Feb.	1	1.58		52.6	......	......	......	2.45	2.50†	......	1.02	......	......	......	......
4	March	1	2.28		53.4	......	......	......	2.83	2.89†	......	1.02	......	......	......	......
5	April	1	0.14		59.0	......	......	......	5.99	6.78	......	1.13	......	......	......	......
6	May	1	0.00		68.3	......	......	......	9.05	10.58	......	1.17	......	......	......	......
7	June	1	0.44		74.5	......	......	......	10.00	11.72	......	1.17	......	......	......	......
8	July	1	0.00		79.4	......	......	......	10.14	12.61	7.3	1.24	0.58	0.72	......	......
9	Aug.	1	0.00*		81.2	......	......	......	9.62	11.27	7.0	1.17	0.62	0.73	......	......
10	Sept.	1	0.00		71.7	......	......	......	7.21	8.46	7.1	1.17	0.84	0.98	......	......
11	Oct.	1	0.63		60.4	......	......	......	4.88	5.42	4.6	1.11	0.85	0.94	......	......
12	Nov.	1	0.16		55.6	......	......	......	2.29	2.44	2.0	1.06	0.82	0.87	......	......
13	Dec.	1	0.76		49.0	......	......	......	1.56	1.50	......	0.96	......	......	......	......
14	Totals	......	6.50		......	......	......	......	67.84	77.99	28.0	......	......	......	......	......
15	Means	......	....		62.7	......	......	......	......	......	......	1.15	0.70	0.82	......	......
(*b*) Austin, Texas (Pans (1) and (4), Standard Dimensions)																
16	Pans					Pan (1)	Pan (4)		Pan (1)	Pan (4)						
17	Jan.	6	2.50		48.0	45.9	47.8	......	2.51	1.92	......	0.77	......	......	87.0	3.2
18	Feb.	8	2.61		52.0	47.9	51.6	......	3.05	2.38	......	0.79	......	......	85.0	3.6
19	March	8	2.10		58.8	53.5	57.8	......	5.33	3.88	......	0.74	......	......	78.4	3.5
20	April	9	4.41		66.2	60.6	65.4	......	6.03	4.92	......	0.83	......	......	81.8	2.5
21	May	9	4.47		73.4	69.0	72.6	......	6.91	5.46	......	0.79	......	......	83.3	1.7
22	June	9	3.51		80.9	75.0	78.7	......	7.86	6.00	......	0.76	......	......	79.9	1.2
23	July	9	2.74		83.2	76.8	80.2	......	8.77	7.07	......	0.81	......	......	78.7	1.3
24	Aug.	9	1.06		83.6	77.2	80.3	......	8.62	6.88	......	0.80	......	......	79.0	1.2
25	Sept.	8	4.22		77.6	72.0	75.7	......	6.39	5.01	......	0.79	......	......	81.3	1.3
26	Oct.	8	3.82		68.2	61.6	66.5	......	5.12	4.03‡	......	0.79	......	......	82.0	1.4
27	Nov.	8	2.29		57.2	52.0	56.4	......	3.03	2.31	......	0.76	......	......	83.6	1.5
28	Dec.	8	1.79		51.0	47.9	50.4	......	2.54	2.01	......	0.79	......	......	83.9	2.8
29	Totals	......	35.52		......	......	......	......	66.16	51.87	......	......	......	......	......	......
30	Means	......	....		66.7	61.6	65.3	......	......	......	......	0.78	......	......	82.0	2.1

* Trace. † Values estimated. ‡ Seven years of record.

(*c*) MORENA RESERVOIR, CALIFORNIA (PANS (3) AND (4), 3 FEET SQUARE AND 1.5 FEET DEEP

31	Pans							Pan (3)	Pan (4)						
32	Jan.	1	0.80	54.6	......	......	......	3.65	3.07	......	0.84	......	......	......	......
33	Feb.	1	1.73	53.2	......	......	......	3.07	2.27	......	0.74	......	......	......	......
34	March	1	0.73	56.9	......	......	......	3.96	3.25	......	0.82	......	......	......	......
35	April	1	0.17	57.2	......	......	......	6.72	6.37	......	0.95	......	......	......	......
36	May	1	0.31	64.3	......	......	......	8.86	7.48	......	0.84	......	......	......	......
37	June	1	0.00	66.3	......	......	......	9.44	8.60	......	0.91	......	......	......	......
38	July	1	0.00	69.9	......	......	......	11.14	10.03	......	0.90	......	......	......	......
39	Aug.	1	0.00	70.0	......	......	......	11.23	9.93	......	0.88	......	......	......	......
40	Sept.	1	0.00	69.5	......	......	......	9.71	8.82	......	0.91	......	......	......	......
41	Oct.	1	0.32	61.8	......	......	......	7.00	5.97	......	0.85	......	......	......	......
42	Nov.	1	1.09	56.8	......	......	......	4.23	3.65	......	0.86	......	......	......	......
43	Dec.	1	2.90	53.0	......	......	......	2.76	......	......	......	......	......	......	......
44	Totals	......	8.05	......	......	......	......	81.77	69.44	......	......	......	......	......	......
45	Means	......	....	61.1	......	......	......	......	......	......	0.88	......	......	......	......

(*d*) BARRETT RESERVOIR, CALIFORNIA (PANS (3) AND (4), 3 FEET SQUARE AND 1.5 FEET DEEP

46	Pans							Pan (3)	Pan (4)						
47	Jan.	1	0.80	54.6	......	......	......	2.32	2.36	......	1.02	......	......	......	......
48	Feb.	1	1.73	53.2	......	......	......	2.67	2.49	......	0.93	......	......	......	......
49	March	1	0.73	56.9	......	......	......	3.61	3.56	......	0.99	......	......	......	......
50	April	1	0.17	57.2	......	......	......	6.06	6.06	......	1.00	......	......	......	......
51	May	1	0.31	64.3	......	......	......	7.53	§	......	§	......	......	......	......
52	June	1	0.00	66.3	......	......	......	9.49	§	......	§	......	......	......	......
53	July	1	0.00	69.9	......	......	......	10.65	§	......	§	......	......	......	......
54	Aug.	1	0.00	70.0	......	......	......	10.30	§	......	§	......	......	......	......
55	Sept.	1	0.00	69.5	......	......	......	9.49	§	......	§	......	......	......	......
56	Oct.	1	0.32	61.8	......	......	......	5.75	5.30	......	0.92	......	......	......	......
57	Nov.	1	1.09	56.8	......	......	......	3.79	3.16	......	0.83	......	......	......	......
58	Dec.	1	2.90	53.0	......	......	......	2.41	2.01	......	0.83	......	......	......	......
59	Totals	......	8.05	......	......	......	......	74.07	24.94	......	......	......	......	......	......
60	Means	......	....	61.1	......	......	......	......	......	......	0.94	......	......	......	......

§ Records lost because pans were out of order.

TABLE 5.—(*Continued*)

Item No.	Month	Number of years of record	PRECIPITATION, IN INCHES		TEMPERATURE, IN DEGREES FAHRENHEIT				TOTAL EVAPORATION FOR THE MONTH, IN INCHES			RATIO OF EVAPORATION			Relative humidity (percentages)	Wind velocity, in miles per hour
					Air	Water						Column (11) to Column (10)	Column (12) to Column (11)	Column (12) to Column (10)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
(*e*) LOWER OTAY RESERVOIR, CALIFORNIA (PANS (3) AND (4), 3 FEET SQUARE AND 1.5 FEET DEEP)																
61	Pans								Pan (3)	Pan (4)						
62	Jan.	1	0.80		54.6	......	......	......	2.09	2.72	......	1.30	......	......	......	......
63	Feb.	1	1.73		53.2	......	......	......	2.45	2.63	......	1.07	......	......	......	......
64	March	1	0.73		56.9	......	......	......	3.92	3.74	......	0.95	......	......	......	......
65	April	1	0.17		57.2	......	......	......	5.93	5.52	......	0.93	......	......	......	......
66	May	1	0.31		64.3	......	......	......	5.66	5.48	......	0.97	......	......	......	......
67	June	1	0.00		66.3	......	......	......	7.80	7.13	......	0.91	......	......	......	......
68	July	1	0.00		69.9	......	......	......	9.98	8.46	......	0.85	......	......	......	......
69	Aug.	1	0.00		70.0	......	......	......	8.87	8.29	......	0.93	......	......	......	......
70	Sept.	1	0.00		69.5	......	......	......	7.00	6.64	......	0.95	......	......	......	......
71	Oct.	1	0.32		61.8	......	......	......	4.99	5.21	......	1.04	......	......	......	......
72	Nov.	1	1.09		56.8	......	......	......	3.65	3.65	......	1.00	......	......	......	......
73	Dec.	1	2.90		53.0	......	......	......	2.18	2.18	......	1.00	......	......	......	......
74	Totals	......	8.05		......	......	......	......	64.52	61.65	......	......	......	......	......	......
75	Means	......	....		61.1	......	......	......	......	......	......	0.96	......	......	......	......
(*f*) INDEPENDENCE, CALIFORNIA (STANDARD, TYPE (4), FLOATING PAN, AND TYPE (3) LAND PAN, 3½ FEET IN DIAMETER AND 4 FEET DEEP; TYPE (3) LAND PAN, 3 FEET SQUARE AND 10 INCHES DEEP)																
76	Pans								Pan (3)	Pan (3)	Pan (4)					
77	Jan.	3	2.14		39.9	......	......	......	2.15‖	2.25¶	1.67	......	0.74	0.78	70	6.5
78	Feb.	3	1.19		39.5	......	......	......	2.72‖	2.25¶	2.42	......	1.08	0.89	64	7.5
79	March	3	0.35		49.1	......	......	......	4.78	6.06‖	4.52	......	0.75	0.95	49	9.2
80	April	3	0.14		56.5	......	......	......	6.66	8.81‖	6.87	......	0.78	1.03	38	8.7
81	May	3	0.01		62.4	......	......	......	7.70‖	10.43‖	8.63	......	0.83	1.12	33	8.1
82	June	2	0.04		72.2	......	......	......	8.20	11.95¶	10.00	......	0.84	1.22	29	7.5
83	July	2	0.12		77.4	......	......	......	8.10	12.55¶	9.45	......	0.75	1.17	30	6.6
84	Aug.	3	0.08		75.7	......	......	......	8.50‖	11.25‖	7.70	......	0.68	0.91	29	6.0
85	Sept.	3	0.47		67.2	......	......	......	7.25‖	8.65‖	6.07	......	0.70	0.84	33	6.8
86	Oct.	3	0.13		57.1	......	......	......	5.08‖	5.70‖	3.87	......	0.68	0.76	41	6.0
87	Nov.	3	0.10		46.7	......	......	......	3.20‖	3.32‖	2.49	......	0.75	0.78	53	5.7
88	Dec.	3	1.43		34.6	......	......	......	2.18‖	1.60‖	1.53	......	0.96	0.70	75	7.0
89	Totals	......	6.21		......	......	......	......	66.52	84.82	65.22	......	......	......	......	......
90	Means	......	....		56.5	......	......	......	......	......	......	......	0.77	0.98	45	7.1

‖ Two-year record. ¶ One-year record.

(*g*) BROOKS RESERVOIR AND LAKE NEWELL, ALBERTA, CANADA (PAN (2) AND PAN (5), 4 FEET IN DIAMETER AND 20 INCHES DEEP)

91	Pans.........		Brooks	Newell		Brooks Reservoir Pan (2)	Brooks Reservoir Pan (5)		Lake Newell Pan (2)	Brooks Reservoir Pan (2)	Brooks Reservoir Pan (5)		Brooks	Newell**		
92	May.........	2	0.83	0.98	......	......	......	......	5.53††	5.74††	......	......	......	0.96	......	3.2
93	June.........	2	2.93	2.74	59	61††	62††	......	5.65††	5.04††	5.86††	......	1.16	1.12	......	2.6
94	July.........	2	1.67	1.62	66	72††	69††	......	6.72	6.50	7.97	......	1.23	1.03	......	2.4
95	Aug..........	2	2.28	2.18	60	65††	64††	......	4.37	4.61	6.19	......	1.34	0.95	......	2.0
96	Sept.........	2	0.55	0.36	54	54††	54††	......	4.08	3.50	4.90	......	1.40	1.16	......	1.6
97	Totals.......	......	8.26	7.88	......	......	......	......	26.35	25.39	24.92	......	......	......	......	......
98	Means.......	......	......	......	59.8	63.0	62.2	......	......	......	......	......	1.27	1.14	......	2.4

(*h*) CHESTNUT HILL RESERVOIR, BOSTON, MASS. (THREE TYPE (5) PANS)

99	Pans:					Pan (5)	Pan (5)	Pan (5)	Pan (5)	Pan (5	Pan (5)					
100	Diameter, in feet.....					1	1	10	1	1	10					
101	Depth, in feet.......					2	8	10	2	8	10					
102	June.........	18‡‡	......	......	......	72.5	72.9	73.3	5.29	5.13	4.58	......	0.89	0.87	......	......
103	July.........	31‡‡	......	......	......	77.2	77.7	77.2	7.99	8.04	7.14	......	0.89	0.89	......	......
104	Aug..........	31‡‡	......	......	......	74.4	75.0	74.4	6.93	7.05	7.41	......	1.05	1.07	......	......
105	Sept.........	30‡‡	......	......	......	65.7	66.6	66.0	5.64	5.81	5.13	......	0.88	0.91	......	......
106	Oct..........	28‡‡	......	......	......	57.0	57.7	56.6	3.39	3.47	2.79	......	0.80	0.82	......	......
107	Totals.......	......	......	......	......	......	......	......	29.24	29.50	27.05	......	......	......	......	......
108	Means.......	......	......	......	......	69.4	70.0	69.5	......	......	......	......	0.92	0.93	......	......

(*i*) CHESTNUT HILL RESERVOIR, BOSTON, MASS. (THREE TYPE (5) PANS)

109	Pans:					Pan (5)	Pan (5)	Pan (5)	Pan(5)§§	Pan(5)§§	Pan(5)§§					
110	Diameter, in feet.....					1	1	10	1	1	10					
111	Depth, in feet.......					2	8	10	2	8	10					
112	June 23–24...	24‖	......	......	64.5	69.4	70.0	69.0	0.490	0.500	0.420	......	0.84	0.86	50.0	11.9
113	June 24–25...	24‖	......	......	70.2	70.0	70.2	70.2	0.320	0.320	0.270	......	0.84	0.84	60.5	7.2
114	July 8–9......	24‖	......	......	77.8	79.6	80.2	79.6	0.320	0.320	0.242	......	0.76	0.76	74.4	6.2
115	July 9–10.....	24‖	......	......	74.4	77.7	78.2	77.8	0.310	0.320	0.255	......	0.80	0.82	76.2	8.0
116	July 10–11....	24‖	......	......	70.6	77.1	77.8	77.3	0.300	0.310	0.280	......	0.90	0.93	73.4	6.8
117	Aug. 7–8.....	24‖	......	......	69.3	74.8	75.2	74.8	0.260	0.260	0.285	......	1.10	1.10	77.4	4.0
118	Sept. 11–12...	24‖	......	......	59.1	66.3	67.0	65.7	0.205	0.210	0.160	......	0.76	0.78	76.9	2.7
119	Oct. 14–15....	24‖	......	......	53.5	55.6	56.0	55.5	0.130	0.140	0.100	......	0.71	0.77	83.8	6.9
120	Totals.......	......	......	......	......	......	......	......	2.335	2.380	2.012	......	......	......	......	......
121	Means.......	......	......	......	67.4	71.3	71.8	71.2	......	......	......	......	0.85	0.86	71.6	6.7

** Pan (5) records for Lake Newell are not listed. †† Records are for 1924 only. ‡‡ Days. ‖ Hours. §§ Evaporation, in inches per 24 hours.

TABLE 6.—SUMMARY OF THE COMPARISON OF THE EVAPORATION FROM DIFFERENT TYPES OF PANS

Item No.	Type of pan	Location	Diameter, in feet	Depth of pan, in feet	Depth of water, in feet	Depth of setting, in feet	Elevation, in feet above sea level	Number of seasons of record	RATIO OF EVAPORATION FROM A GIVEN PAN OR RESERVOIR TO EVAPORATION FROM:						Observer
									Class A land pan, Type (1)	Sunken land pan, Type (2)	Colorado sunken land pan, Type (3)	Sunken circular land pan, Type (2)	U. S. Geological Survey floating pan, Type (4)	U. S. Weather Bureau floating pan Type (5)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1	(2)	Denver, Colo.	1.0	3.0	2.75	2.75	5 346	3	1.12	1.49	1.26	......	......	......	Sleight
2	(2)	Denver, Colo.	2.0	3.0	2.75	2.75	5 346	2	0.90	1.21	1.02	......	......	......	"
3	(2)	Denver, Colo.	3.39	3.0	2.75	2.75	5 346	3	0.85	1.13	0.96	......	......	......	"
4	(2)	Denver, Colo.	6.0	3.0	2.75	2.75	5 346	3	0.77	1.02	0.87	......	......	......	"
5	(2)	Denver, Colo.	9.0	3.0	2.75	2.75	5 346	2	0.71	0.95	0.80	......	......	......	"
6	(2)	Denver, Colo.	12.0	3.0	2.75	2.75	5 346	3	0.70	0.94	0.80	......	......	......	"
7	(2)	Denver, Colo.	2.0	0.5	0.25	0.25	5 346	2	0.90	1.20	1.02	......	......	......	"
8	(2)	Denver, Colo.	2.0	1.0	0.75	0.75	5 346	2	0.89	1.19	1.01	......	......	......	"
9	(2)	Denver, Colo.	2.0	1.5	1.25	1.25	5 346	2	0.90	1.20	1.02	......	......	......	"
10	(2)	Denver, Colo.	2.0	2.0	1.75	1.75	5 346	2	0.90	1.20	1.02	......	......	......	"
11	(2)	Denver, Colo.	2.0	6.0	5.75	5.75	5 346	2	0.90	1.19	1.01	......	......	......	"
12	(2)	Denver, Colo.	6.0	1.0	0.75	0.75	5 346	1	0.75	1.00	0.85	......	......	......	"
13	(2)	Denver, Colo.	6.0	2.0	1.75	1.75	5 346	1	0.75	1.00	0.85	......	......	......	"
14	(3)	Denver, Colo.	2.0*	3.0	2.75	2.75	5 346	2	0.92	1.23	1.04	......	......	......	"
15	(3)	Denver, Colo.	3.0*	3.0	2.75	2.75	5 346	3	0.88	1.18	1.00	......	......	......	"
16	(1)	Denver, Colo.	2.0	0.83	0.62	0.00§	5 346	1	1.19	1.58	1.34	......	......	......	"
17	(1)	Denver, Colo.	4.0	0.83	0.62	0.00§	5 346	3	1.00	1.33	1.13	......	......	......	"
18	(1)	Denver, Colo.	6.0	0.83	0.62	0.00§	5 346	1	0.94	1.25	1.06	......	......	......	"
19	(5)	Salton Sea, Calif.	2.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	1.02	Bigelow
20	(5)	Salton Sea, Calif.	4.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	1.00	"
21	(5)	Salton Sea, Calif.	6.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	0.87	"
22	(5)	Salton Sea, Calif.	12.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	0.82	"
23	(1)	Salton Sea, Calif.	2.0	0.83	0.6	0.00§	—200	4¶	1.21	......	......	......	......	......	"
24	(1)	Salton Sea, Calif.	4.0	0.83	0.6	0.00§	—200	4¶	1.00	......	......	......	......	......	"
25	(1)	Salton Sea, Calif.	6.0	0.83	0.6	0.00§	—200	4¶	0.92	......	......	......	......	......	"
26	(4)	Independence, Calif.	3.0*	0.83	0.67	0.5	3 907	3	......	......	0.97**	......	......	......	Lee
27	(4)	Santa Clara Valley, Calif.	3.0*	1.0	0.83	......	......	2	......	......	0.71††	......	......	......	Duryea
28	(5)	Brooks Reservoir, Alberta, Canada	4.0	1.67	1.33	1.33	2 450	2	......	......	......	1.27	......	......	O'Neil
29	(5)	Brooks Reservoir, Alberta, Canada	4.0	1.67	1.33	1.33	2 450	2	......	......	......	1.14‡‡	......	......	"
30	(‡)	Newell Reservoir, Alberta, Canada	16.430†	......	......	......	2 485	7	......	......	......	0.95	0.70	......	"

* Square. † Acres. ‡ Lake or reservoir. § Exposed. ‖ Well immersed. ¶ Months. ** Sunken land pan, Column (12), 10 in. deep. †† Sunken land pan, Column (12), 12 in. deep; floating pan on raft in reservoir; ratio, average of six land pans to three floating pans. ‡‡ At Lake Newell, Alberta, Canada.

31	(‡)	Buena Vista Lake, Calif. .	10 000†	......	......	......	420	1	......	......	0.82	......	0.81	......	Harding
32	(‡)	Lake Elsinore, Calif.	5 500†	......	......	......	1 261	1	......	......	......	......	......	......	"
33	(‡)	Lake Hodges, Calif.	1 317†	......	......	......	315	0.5	......	......	0.96	......	1.08	......	Lippincott
34	(‡)	Cuyamaca Lake, Calif. . .	978†	......	......	......	4 600	2¶	......	......	......	......	......	......	"
35	(‡)	Cuyamaca Lake, Calif. . .	978†	......	......	......	4 600	2¶	......	......	1.19	......	......	......	"
36	(2)	Stonyford, Calif.	4.0	3.0	2.75	2.75	1 200	1¶	0.88	......	0.96	1.00	......	1.01	Rohwer
37	(2)	Los Griegos, N. Mex.	4.0	2.0	1.75	1.75	4 970	2	0.71	......	......	......	......	......	Houk
38	(2)	Garnett, Colo.	4.0	2.0	......	1.83	7 700	3	0.78	......	......	......	......	......	Tipton
39	...	Means, Items No. 36, 37, and 38.	......	......	......	......	......	......	0.79	......	0.96	1.00	......	1.01	....
40	(2)	Denver, Colo.	6.0	1.0	0.75	0.75	5 346	3	0.75	1.00	0.85	......	......	......	Sleight
41	(2)	Denver, Colo.	6.0	2.0	1.75	1.75	5 346	1	0.75	1.00	0.85	......	......	......	"
42	(2)	Denver, Colo.	6.0	3.0	2.75	2.75	5 346	1	0.77	1.02	0.87	......	......	......	
43	...	Means, Items Nos. 40, 41, and 42.	......	......	......	......	......	......	0.76	1.01	0.86	......	......	......	....
44	(2)	Lincoln, Nebr.	7.5	2.0	1.67	2.0	1 150	4	0.71	......	......	......	......	......	Loveland
45	(2)	Denver, Colo.	9.0	3.0	2.75	2.75	5 346	2	0.71	0.95	0.80	......	......	......	Sleight
46	(2)	Denver, Colo.	12.0	3.0	2.75	2.75	5 346	3	0.70	0.94	0.80	......	......	......	"
47	(2)	Milford Utah, ...	12.0	3.0	......	2.75	4 962	3	0.67	......	......	......	......	......	White
48	(2)	Fort Collins, Colo.	85.0	6.75	6.4	6.4	5 000	3	0.70	......	0.79	......	0.77	......	Rohwer
49	(‡)	East Park Reservoir, Stonyford, Calif.	1 800†	......	......	......	1 200	1¶	0.69	......	0.75	0.78	......	0.78	"
50	...	Means, Items Nos. 44 to 49	......	......	......	......	......	......	0.70	0.94	0.78	0.78	0.77	0.78	....
51	(3)	Stonyford, Calif.	3.0*	1.5	1.3	1.4	1 200	1¶	0.91	......	1.00	1.04	......	1.04	Rohwer
52	(3)	Fort Collins, Colo.	3.0*	4.5	1.33	1.33	5 000	3	0.89	......	1.00	......	0.98	......	"
53	(3)	Denver, Colo.	3.0*	3.0	2.75	2.75	5 346	3	0.88	1.18	1.00	......	......	......	Sleight
54	...	Means, Items Nos. 51, 52, and 53.	......	......	......	......	......	......	0.89	1.18	1.00	1.04	0.98	1.04	....
55	(1)	Stonyford, Calif.	4.0	0.83	0.6	0.00§	1 200	1¶	1.00	......	1.10	1.14	......	1.14	Rohwer
56	(1)	Fort Collins, Colo.	4.0	0.83	0.62	0.00§	5 000	3	1.00	......	1.13	......	1.11	......	"
57	(1)	Denver, Colo.	4.0	0.83	0.62	0.00§	5 346	3	1.00	1.33	1.13	......	......	......	Sleight
58	...	Means, Nos. 55, 56, and 57	......	......	......	......	......	......	1.00	1.33	1.12	1.14	1.11	1.14	....

* Square. † Acres. § Exposed. ¶ Months.

TABLE 6.—(*Continued*)

									RATIO OF EVAPORATION FROM A GIVEN PAN OR RESERVOIR TO EVAPORATION FROM:						
Item No. (1)	Type of pan (2)	Location (3)	Diameter, in feet (4)	Depth of pan, in feet (5)	Depth of water, in feet (6)	Depth of setting, in feet (7)	Elevation, in feet above sea level (8)	Number of seasons of record (9)	Class *A* land pan, Type (1) (10)	Sunken land pan, Type (2) (11)	Colorado sunken land pan, Type (3) (12)	Sunken circular land pan, Type (2) (13)	U. S. Geological Survey floating pan, Type (4) (14)	U. S. Weather Bureau floating pan, Type (5) (15)	Observer (16)
59	(4)	Buena Vista, Calif.......	3.0*	2.0	1.75	1.5	420	1	......	......	1.15	......	1.00	......	Harding
60	(4)	Austin, Tex............	3.0*	1.5	......	......	475	9	0.78	......	......	......	1.00	......	Geological Survey.
61	(4)	Lower Otay Reservoir, Calif...............	3.0*	1.5	1.25	......	500	1	......	......	0.96	......	1.00	......	Savage
62	(4)	Barrett Lake, Calif......	3.0*	1.5	1.25	......	1 600	1	......	......	0.94	......	1.00	......	"
63	(4)	Morena Lake, Calif......	3.0*	1.5	1.25	......	3 000	1	......	......	0.88	......	1.00	......	"
64	(4)	Fort Collins, Colo.......	3.0*	1.5	1.25	1.25	5 000	3	0.91	......	1.02	......	1.00	......	Rohwer
65	(4)	Denver, Colo...........	3.0*	1.5	1.25	1.25§§	5 346	2	......	......	......	......	1.00	1.09	Sleight
66	...	Means, Items Nos. 59 to 65.................	......	......	......	......	......	......	0.84	......	0.99	......	1.00	1.09	....
67	(5)	Stonyford, Calif.........	4.0	0.83	0.5	......	1 200	1¶	0.88	......	0.96	1.00	......	1.00	Rohwer
68	(5)	Nelson Reservoir, Mont..	4.0	0.83	0.5	0.5	2 215	4	0.68	......	......	......	......	1.00	Reclamation Bureau.
69	(5)	Fall River Mills, Calif ...	4.0	0.67	0.5	0.33	3 340	5	0.76	......	......	......	......	1.00	Pac. Gas & Elec. Co.
70	(5)	Denver, Colo...........	4.0	1.0	0.75	0.75	5 346	1	......	......	......	......	0.92	1.00	Sleight
71	...	Means, Items Nos. 67 to 70.................	......	......	......	......	......	......	0.77	......	0.96	1.00	0.92	1.00	....
72	(5)	Salton Sea, Calif........	2.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	1.01	Bigelow
73	(5)	Denver, Colo...........	2.0	1.0	0.75	0.75	5 346	1	......	......	......	......	1.00	1.08	Sleight
74	...	Means, Items Nos. 72 and 73.................	......	......	......	......	......	......	......	......	......	......	1.00	1.04	....
75	(5)	Salton Sea, Calif........	6.0	0.83	0.6	‖	—210	......	......	......	......	......	......	0.87	Bigelow
76	(5)	Denver, Colo...........	6.0	1.0	0.75	0.75	5 346	......	......	......	......	......	0.86	0.94	Sleight
77	...	Means, Items Nos. 75 and 76.................	......	......	......	......	......	......	......	......	......	......	0.86	0.90	....

* Square. ‖ Well immersed. §§ Assumed to be standard depth. ¶ Months.

Tables 5(h) and 5(i) are from observations made[17] in 1885, on floating pans of the same type, but of different diameters and depths. All pans were floated in Chestnut Hill Reservoir, Boston, Mass. (Elevation 129), supported by a raft 20 ft wide and 40 ft long. They were filled with water to within 3 in. of their tops and submerged to within 6 in. of their tops (as scaled from photographs). Readings were taken daily for Table 5(h) and hourly for Table 5(i). The anemometer was 30.5 ft above the water surface.

Summary.—Table 6 is a summary tabulation of conclusions drawn from the preceding tables.

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[17] *Transactions,* Am. Soc. C. E., Vol. XV (1886), p. 581.

EVAPORATION FROM RESERVOIR SURFACES

By Robert Follansbee,[18] M. Am. Soc. C. E.

Synopsis

This paper contains the results of all available evaporation records, not only in the United States and outlying possessions, but also in foreign countries, reduced to reservoir surface evaporation by means of coefficients which are stated for each record. These results are given in summarized form, together with records of temperature, wind velocity, and relative humidity, as far as available. A total of 210 evaporation records are presented.

After the summary the relative effect of temperature, wind velocity, and relative humidity is shown by comparison between pairs of records in which two factors are the same and the third is widely different.

A brief discussion of the variation in evaporation throughout the United States concludes the paper.

Introduction

For many years records of evaporation have been kept by different organizations, which have used pans varying not only in size and shape, but in setting. Few, if any, of these records give directly the evaporation from reservoir surfaces, but must be corrected by coefficients that vary with the size of the pan and its setting. By reducing the records to reservoir surface evaporation, the data in which engineers are chiefly interested will be obtained, and a common basis for comparison of evaporation under varying climatic conditions will be available. The writer has compiled, not only for the United States and its possessions, but also for foreign countries, all evaporation records of which he has knowledge, and with them the available related data on temperature, relative humidity, and wind velocity.

Coefficients to Reduce Observed Evaporation to Reservoir Surface

To reduce the observed evaporation to reservoir surface evaporation, use has been made of Mr. Rohwer's paper, published as a part of this Symposium, which brings together the results of many experiments on the subject.

Of the many evaporation records available, the greater number have been taken by pans of the following types:

(1) The United States Weather Bureau Class *A* pan, which is a circular pan, 4 ft in diameter and 10 in. deep, placed on an open timber platform resting on the ground.

NOTE.—This paper is published with the approval of the Director of the U. S. Geological Survey.

[18] Dist. Engr., U. S. Geological Survey, Denver Colo.

(2) The United States Bureau of Plant Industry pan, which is a circular pan, 6 ft in diameter and 2 ft deep, sunk in the ground with its rim 3 in. above the surface.

(3) The floating pan, which may be either 3 ft square and 18 in. deep, or a circular pan, 4 ft in diameter and 10 in. deep, suspended from an enclosing raft with some form of wooden shield to protect the water in the pan from wave action.

United States Weather Bureau Class A Pan.—In addition to the records cited by Mr. Rohwer, those for the Salton Sea, in California, have been used for an approximate comparison. The work done at Salton Sea during 1909 and 1910 does not lend itself to an exact comparison between evaporation from large water surfaces and evaporation from land pans of the Class *A* type, but the following rough approximation can be made.

A Class *A* pan was placed on a platform 2 ft above the water surface at a point in Salton Sea 7 500 ft from shore. This setting may be considered to approximate that of a Class *A* pan on a wooden platform on the ground. One year's record from this pan showed an evaporation of 106.45 in. The actual evaporation from Salton Sea during that period has been computed variously as 69 to 75 in. With a mean value of 72 in., the coefficient for the Class *A* pan is 72 ÷ 106.45 = 0.68.

Another Class *A* pan was placed on a platform 2 ft above the water surface, at a point in Salton Sea 500 ft from shore. This showed an evaporation of 108.65 in. For this pan the coefficient was 72 ÷ 108.65 = 0.66. The mean relative humidity was 60 per cent.

Table 7 is a summary of Mr. Rohwer's results as well as those at Salton Sea; and of the work of the late R. B. Sleight, Assoc. M. Am. Soc. C. E., at Denver, Colo. In view of the results indicated, the coefficient to reduce the U. S. Weather Bureau Class *A* pan records to reservoir surface evaporation has been taken by the writer as 0.69.

TABLE 7.—COEFFICIENTS TO REDUCE CLASS *A* PAN RECORDS TO RESERVOIR SURFACE EVAPORATION

Location	Years of record	Comparison with	Coefficient	Temperature, in degrees Fahrenheit	Wind velocity, in miles per hour	Relative humidity (percentages)
Denver, Colo.	1915–17	12-ft ground pan	0.70	....	....	64
Fort Collins, Colo.	1926–28	85-ft reservoir	0.70	47	1.6	68
East Park Reservoir, California	1930	1 800-acre reservoir	0.68	80	1.7	47
Milford, Utah	1925–27	12-ft ground pan	0.70	50	4.3	41
Lincoln, Nebr.	1917–20	7½-ft ground pan	0.67	51	4.1	65
Salton Sea, California	1909–10	Indirect method	0.68	....	....	60
Salton Sea, California	1909–10	Indirect method	0.66	....	....	60

It should be noted that the experiments were made under climatic conditions having a range of relative humidity from 41% to 68%, a range in temperature from 47° to 80° F, and a range in wind velocity from 1.6 to 4.3 miles per hour. As an increase of more than one-half in relative humidity and similar ranges in temperature and wind velocities showed little change in the coefficient to be applied to Class *A* pans, it appears that for an increase in relative humidity to 70% or 80%, such as is found in the eastern part of the United States, the coefficient will be substantially the same.

United States Bureau of Plant Industry Pans.—The only direct comparison between a Bureau of Plant Industry pan and a large water surface was that made by Sleight in the Denver experiments, in which he found the coefficient to be 0.94.[19]

Floating Pans.—Theoretically, records from a floating pan of galvanized iron should give closely and consistently the evaporation from the water surface in which it floats. Practically, however, it has been found difficult to maintain accurate records, owing to the action of waves, which causes water to enter the pan or to spill out of it. Furthermore, a constant factor tending to increase the observed evaporation is the wetted strip around the inside of the pan just above the normal water surface, produced by wave action within the pan itself. To show the inconsistency of the results of experiments on floating pans, it is only necessary to reduce the records presented by Mr. Rohwer (to which have been added a few compiled by the writer) to the common basis of reservoir evaporation.

Direct comparisons between (1) floating pans, 3 ft square and 18 in. deep, and (2) floating circular pans, 4 ft in diameter and 10 in. deep, with large water surfaces, show the following results:

Location	Square Pan	Circular Pan
Denver, Colo.	0.88	....
Fort Collins, Colo.	0.77	....
East Park Reservoir, California	....	0.78
Lake Elsinore, California	0.84	....
Buena Vista Lake, California	0.70	....
Upper Otay Reservoir, California	0.87	....

Indirect comparisons by means of records from Class *A* pans give the following results:

Location	Square Pan	Circular Pan
Oakdale, Calif.	....	0.92
Fall River Mills, Calif.	....	0.52
Austin, Tex.	0.89	....
Granite Reef, Ariz.	....	0.81

These inconsistent results are practically the same for both types of floating pans, and an average of all coefficients (except that for Fall River Mills), gives 0.83 as the factor to reduce records from floating pans of the two types considered to evaporation from large water surfaces.

Egyptian Records.—The Egyptian Government has maintained evaporation records at points in the Nile Valley from Alexandria to the Southern Sudan. These records have been taken both by floating pans and Piche evaporimeters used in a meteorological screen. To determine the reduction factors, comparisons between Piche tubes and both floating and land pans were made. As it was not found practicable to experiment with pans larger than 2 m square, that size was used as the best available approximation to an open water surface. At Helwan Observatory, on the hills overlooking the Nile Valley, about 15 miles from Cairo, the factor to reduce the Piche records to a 2-m land pan was found to be 0.66. At Abbassiya, near Cairo, the factor to reduce Piche records to a 2-m floating pan square was 0.50, and, for a 1-m floating pan, the factor was 0.88.

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[19] "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 308.

By considering the relation between floating and land pans as given by Sleight[20] in his first report on the Denver experiments and combining these with the Helwan observations, the factor to reduce Piche records to those from a 1-m floating-pan record was 0.59. At Abbassiya, the corresponding factor was 0.57. The engineers of the Egyptian Government, therefore, used a factor of 0.50 for the Piche tubes; for their 1-m floating-pan records they used 0.88. In so doing, it was stated that the evaporation would not be underestimated. The factors were determined for each month of the year and there was no evidence of any seasonal variation, the probable departure of the monthly ratios from the mean of the year being 3% for the pans and 10% for the Piche tubes.[21]

Turkestan Records.—The late Lyman E. Bishop, M. Am. Soc. C. E., who spent two years in Turkestan, furnished evaporation records taken by the Soviet Government since 1920 at experiment stations located at Fergana, Andidjan, Khodjent, and Osh, in the Fergana Valley, in Southern Turkestan. This is a region with an irrigated area of 1 600 000 acres and an additional area of 750 000 acres of seeped land. Although no actual records are available, the temperatures are stated to increase gradually from about 5° F, in January, to 120° to 130° F, in July and August, with a monthly mean for these months of about 95 degrees.

The annual rainfall is from 10 to 12 in., all of which occurs between November and March. It is a region of little wind, and as the valley is almost enclosed by high mountains, the evaporation from the irrigated land on which a large quantity of water is used, and from the seeped land, causes a humid atmosphere, particularly at the evaporation pans.

The evaporation records were taken by pans about 6.5 ft square and 3 ft deep set in the ground. By comparison with Sleight's experiments at Denver, the coefficient to reduce the records to reservoir surface evaporation has been taken as 0.95.

REDUCTION OF PAN RECORDS TO RESERVOIR SURFACE EVAPORATION

The foregoing coefficients (as well as some others determined from Sleight's experiments), which have been used by the writer, are based on mean values covering several months, and are shown by the experiments themselves to vary from month to month. For some types of pans, furthermore, the mean coefficient itself may be considerably in error, as shown by the inconsistent results. It is with these facts in mind that the observed

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[20] *Journal of Agricultural Research,* Vol. X, No. 5, July 30, 1917.

[21] "The Nile Basin," Vol. 1, Cairo, Egypt, Ministry of Public Works, 1931.

TABLE 8.—SUMMARY OF EVAPORATION RECORDS REDUCED TO RESERVOIR SURFACE EVAPORATION

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	WIND VELOCITY			Relative humidity (percentage)	RESERVOIR SURFACE EVAPORATION, IN INCHES			Percentage range of annual evaporation	Coefficient for pan
				Near-by U. S. Weather Bureau Station		At the pan, in miles per hour		April–September	Annual	Maximum per month		
				Height, in feet	Velocity, in miles per hour							
(*a*) ATLANTIC SEABOARD												
Gardiner, Me.	100	1915–24	45	.....	.....	2.2	..	18.10	24.26	4.56	92–111	0.69
Voorheesville, N. Y.	330	1917–30	48	115	7.5	1.9	74	19.66	25.98	4.71	92–107	0.69
Ithaca, N. Y.	800	1918–30	47	100	9.1	1.8	78	17.11	22.54	4.17	90–114	0.69
Runyon, N. J.	50	1924–30	52	183	11.0	2.5	71	19.78	27.85	4.88	92–106	0.69
Pleasantville, N. J.	40	1924–30	51	172	16.5	2.9	74	23.02	31.81	5.40	84–104	0.69
Washington, D. C.	280	1915–17	54	85	6.5	2.3	69	23.52	34.53	4.87	97–103	0.69
Chapel Hill, N. C.	500	1921–30	61	.....	.....	1.1	69	20.03	28.56	4.71	93–109	0.69
Birmingham, Ala.	650	1910	63	.....	7.0	....	72	32.18	42.99	6.25	......	0.83
(*b*) GREAT LAKES												
Grand River Lock, Wis.	780	1905–12	46	.....	.....	....	..	21.64	28.57	5.75	85–105	0.83
Menasha, Wis.	740	1906–12	47	.....	.....	....	..	12.29	17.54	3.88	73–115	0.83
Madison, Wis.	860	1906–11	46	78	10.0	....	75	12.91	19.82	3.04	97–102	0.83
Wooster, Ohio	1 000	1916–29	49	.....	.....	2.3	..	18.25	24.85	5.21	85–112	0.69
(*c*) MISSISSIPPI VALLEY												
Centreville, Minn.	880	1919–27	45	.....	.....	4.0	..	24.11	30.97	7.37	97–107	0.69
Iowa City, Iowa	610	1907–10	46	.....	.....	....	..	19.59	30.08	5.36	93–112	0.83
Columbus, Ohio	763	1918–30	52	230	10.7	2.0	74	19.21	26.81	4.94	89–108	0.69
Cincinnati, Ohio	520	1910	54	51	6.8	....	68	29.46	38.19	6.02	......	0.83
Clarksburg, W. Va.	1 030	1923–30	53	.....	.....	2.6	79	20.65	26.60	6.05	94–117	0.69
Columbia, Mo., No. 1	750	1916–27	54	84	8.0	1.5	71	20.28	28.13	5.31	88–114	0.69
Columbia, Mo., No. 2	750	1916–26	54	84	8.0	2.9	71	26.31	35.82	6.85	......	0.69
(*d*) GULF COAST												
Silverhill, Ala.	250	1918–30	67	161	7.3	1.8	78	25.35	38.27	5.73	90–111	0.69
Crowley, La.	21	1910–19	68	.....	.....	2.8	77	30.76	46.90	7.18	96–106	0.94
Beeville, Tex.	214	1922–30	70	.....	.....	5.7	..	37.64	56.57	10.64	91–114	0.95
(*e*) NORTH DAKOTA												
University, Grand Forks	820	1905–20	38	58	9.8	....	79	21.72	27.07	5.82	90–117	0.83
Williston	1 875	1909–16	39	48	8.3	5.5	74	32.12	38.79	8.45	87–109	0.94
Dickinson	2 543	1907–20	40	.....	.....	6.3	..	31.19	39.21	9.38	85–130	{0.94 {0.95
Mandan	1 750	1914–20	40	57	10.0	5.6	72	32.08	39.80	8.62	93–112	0.94
Edgeley	1 468	1907–20	40	56	13.9	7.0	77	27.92	35.90	7.84	89–114	{0.94 {0.95
Hettinger	2 253	1911–20	41	.....	.....	6.2	75	31.04	39.60	10.48	82–127	0.94
(*f*) SOUTH DAKOTA												
Newell	2 880	1908–29	45	.....	.....	6.6	..	33.63	44.63	10.08	83–123	0.94
Rapid City	3 240	1916–21	46	.....	.....	2.0	58	25.61	36.43	7.02	92–112	0.69
Ardmore	3 557	1913–20	44	.....	.....	6.4	62	35.40	46.59	9.11	83–113	0.94
(*g*) NEBRASKA												
Lincoln	1 250	{1895–1910 {1917–20	52	81	10.7	4.1	70	33.89	43.53	11.66	76–113	0.95
Lincoln	1 250	1917–30	51	81	10.0	4.1	69	32.06	42.04	9.92	87–118	0.69
North Platte A	3 000	1907–30	48	51	9.2	7.5	68	38.58	46.31	11.84	85–123	{0.94 {0.95
North Platte B	2 820	1917–30	48	51	9.2	5.7	69	35.48	43.11	8.86	90–108	0.94
Mitchell (Lake Minatare)	4 120	1918–21	47	6	8.4	....	..	27.13	33.60	6.75	......	0.69
Mitchell (Sunflower)	4 070	1909–17	47	6	7.9	....	68	27.55	34.95	7.75	......	0.69
Mitchell	4 080	1911–29	47	.....	.....	7.0	..	34.17	41.78	9.28	83–118	0.94

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity			Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches			Percentage range of annual evaporation	Coefficient for pan
				Near-by U. S. Weather Bureau Station: Height, in feet	Near-by U. S. Weather Bureau Station: Velocity, in miles per hour	At the pan, in miles per hour		April-September	Annual	Maximum per month		
(*h*) Kansas												
Lawrence	825	1916–19	55	.....	.....	4.4	..	31.25	44.80	7.62	......	0.69
Manhattan	1 010	1924–29	54	.....	.....	3.4	..	30.14	42.65	8.06	91–112	0.69
Wichita	1 300	1918–27	56	158	13.0	4.2	68	34.60	49.62	9.27	92–113	0.69
Hays	2 000	1907–29	53	.....	.....	8.0	65	43.67	61.97	12.25	81–122	0 94 0.95
Colby	3 135	1914–30	51	.....	.....	6.3	64	38.86	54.43	10.77	83–114	0.94
Garden City	2 830	1908–30	54	.....	.....	8.0	60	49.97	68.27	13.08	83–111	0.94 0.95
Tribune	3 543	1918–30	51	.....	.....	6.1	..	42.57	57.35	11.53	88–107	0.69
(*i*) Oklahoma												
Lake Lawtonka	....	1913–17	60	.....	.....	....	..	37.27	49.45	9.82	......	0.83
Lawton	1 111	1916–20	61	.....	.....	6.4	..	41.75	54.35	11.42	87–113	0.94
Woodward	1 900	1914–20	57	.....	.....	8.2	..	45.73	58.53	10.84	89–108	0.94
(*j*) Texas												
Austin	475	1916–29	68	148	7.5	2.0	68	28.72	42.47	8.49	77–116	0.83
Austin	475	1916–30	68	148	7.5	2.0	68	30.77	45.83	8.50	82–125	0.69
San Antonio	700	1907–12	70	300	6.8	....	68	43.46	63.71	10.14	93–109	0.95
Dilley	586	1928–30	71	.....	.....	2.6	65	39.63	57.02	9.16	......	0.69
Laredo	455	1917–18	73	.....	.....	3.8	64	44.44	67.47	9.39	......	0.69
Dalhart	4 000	1908–20	54	.....	.....	7.9	61	50.67	67.78	12.04	89–108	0.94 0.95
Spur	.....	1922–30	61	.....	.....	6.0	61	42.84	62.44	10.80	81.114	0.95
Chillicothe	1 406	1912–19	63	.....	.....	7.2	60	46.91	68.57	12.20	86–114	0.94
Amarillo	3 680	1907–19	56	49	13.0	9.6	56	49.00	66.00	12.63	84–109	0.94
Big Springs	2 396	1915–20	63	.....	.....	6.7	59	55.01	75.65	12.56	91–109	0.94
Balmorhea	....	1930–31	64	.....	.....	4.4	..	40.31	59.07	7.55	......	0.95
El Paso	3 700	1889–93	64	175	8.3	....	36	49.95	71.16	10.79	......	0.83
(*k*) Montana												
Malta (Nelson Reservoir)	2 215	1917–23	42	.....	.....	5.7	..	30.67	36.85	.....	......	0.83
Malta (Nelson Reservoir)	2 215	1926–30	42	.....	.....	5.7	..	31.18	37.07	10.62	73–120	0.69
Huntley	2 930	1911–30	46	.....	.....	4.3	..	30.98	42.21	8.77	89–119	0.94
Moccasin	4 228	1909–20	41	.....	.....	6.2	69	31.58	42.31	9.40	88–114	0.94
Havre	2 505	1916–20	41	44	7.8	5.7	66	33.56	42.13	8.88	89–109	0.94
Bozeman	4 754	1918–30	41	.....	.....	2.7	..	24.72	33.77	7.58	85–117	0.69
Valier	....	1916–30	41	.....	.....	3.2	..	26.60	35.89	7.74	86–112	0.69
Willow Creek Reservoir	4 130	1919–28	44	.....	.....	7.8	..	26.06	37.20	7.62	92–114	0.83
Sun River Canyon	4 474	1919–22	42	.....	.....	10.7	..	29.47	40.14	6.08	......	0.83
St. Ignatius	3 003	1922–23	45	.....	.....	....	..	26.84	36.98	5.29	86–121	0.75
(*l*) Wyoming												
Archer	6 012	1913–20	45	101	13.3	8.7	56	34.27	47.65	8.06	88–112	0.90
Sheridan	3 790	1917–20	44	47	5.5	4.3	..	33.28	43.33	9.69	89–118	0.94
Shoshone Reservoir	5 390	1916–29	46	.....	.....	....	..	31.70	43.27	9.46	86–119	0.74
Powell	4 390	1911–19	43	.....	.....	....	..	24.36	32.40	.....	......	0.76
Ralston	4 600	1911–16	44	.....	.....	....	..	31.80	40.98	.....	......	0.76
Pathfinder	5 900	1918–30	44	.....	.....	....	..	31.00	40.74	8.62	92–116	0.66
Laramie	7 150	1891–95	40	.....	.....	....	..	30.52	40.71	5.97	94–108	0.89
(*m*) Colorado												
Akron	4 650	1908–20	48	.....	.....	7.4	62	39.26	53.95	10.61	84–110	0.94 0.95
Lamar	3 900	1903–04	54	.....	.....	....	..	35.85	52.54	.....	......	0.83
Fort Collins	4 998	1887–1927	46	.....	6.1	1.7	68	30.41	42.19	6.12	81–119	0 80
Fort Collins	4 998	1927–28	47	.....	6.1	1.6	68	25.55	35.06	6.28	......	1.00
Denver	5 340	1916	49	.....	7.8	4.5	64	38.09	52.15	8.20	......	0.99
Pueblo	4 790	1908–09	51	.....	6.8	....	..	34.98	47.82	6.73	......	0.83
Wagonwheel Gap	9 610	1920–24	34	.....	.....	2.0	64	16.12	22.32	3.22	94–110	0.69

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity			Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches			Percentage range of annual evaporation	Coefficient for pan
				Near-by U. S. Weather Bureau Station		At the pan, in miles per hour		April-September	Annual	Maximum per month		
				Height, in feet	Velocity, in miles per hour							
(*n*) New Mexico												
Tucumcari	4 194	1913–20	57	. ...	.....	6.1	50	51.51	73.56	12.89	89–114	0.94
Avalon Reservoir	3 188	1914–24 '29	63	85	7.8	....	52	42.15	60.73	10.80	90–112	0.83
Carlsbad	3 100	1910	64	.....	.....	....	..	48.11	74.01	8.87	......	0.69
Carlsbad	3 100	1910	64	.....	.....	....	..	40.60	65.10	7.65	......	0.69
Therma	8 200	1929–30	43	.....	.....	3.6	..	30.07	39.17	7.60	......	0.69
Santa Fe	7 010	1917–30	48	53	6.1	2.6	54	33.66	44.82	8.46	87–117	0.69
Farmington	5 300	1914–29	57	.....	.....	....	..	32.02	43.66	9.95	85–114	0.83
Los Griegos	4 970	1926–28	54	.....	.....	3.2	49	39.65	54.79	8.28	......	0.69
Elephant Butte	4 265	1916–30	60	.....	.....	3.9	..	47.24	66.99	11.75	87–114	0.69
Las Cruces	3 683	1918–30	60	50	6.9	1.9	41	41.15	59.46	9.38	88–114	0.69
Deming	4 300	1914–29	68	.....	.....	....	..	34.53	54.48	9.94	67–123	0.83
(*o*) Idaho												
Mud Lake	4 784	1921–23	40	.....	.....	4.4	..	30.75	39.30	7.81	......	0.83
Milner	4 200	1927–30	48	.....	.....	2.3	..	28.60	39.91	7.41	95–105	0.69
Arrowrock	3 220	1916–30	49	.....	.....	0.8	..	29.32	40.09	8.07	92–115	0.69
Aberdeen	4 400	1912–19	44	68	8.7	6.0	58	39.39	50.86	12.10	94–111	0.94
Jerome	3 893	1919–27	49	.....	.....	2.7	..	26.96	37.89	7.25	82–114	0.69
Deer Flat	2 510	1916–25	48	.....	....	2.4	57	26.14	38.59	6.12	89–107	0.69
(*p*) Utah												
Myton	5 030	1918–30	46	.....	.....	2.9	..	33.49	43.72	9.95	93–110	0.69
Utah Lake Outlet	4 497	1901–06	50	.....	.....	....	51	32.87	40.70	9.30	......	0.80
West Portal, Strawberry Tunnel	7 500	1910–15	41	.....	.....	....	..	20.81	26.12	.....	......	0.78
East Portal, Strawberry Tunnel	7 600	1905–14, 15	36	.....	.....	....	..	15.73	19.72	.....	......	0.78
Spanish Fork	4 700	1911–15	51	.....	.....	....	55	30.23	40.10	10.47	......	0.78
Provo	4 650	1918–26	49	203	7.1	1.2	54	24.10	30.99	5.96	94–103	0.69
Provo	....	1910–17	49	203	7.1	....	..	20.81	27.29	.....	......	0.78
Utah Lake	4 497	1923–30	48	.....	.....	4.2	53	34.80	43.19	8.67	93–111	0.69
Salt Lake City	4 250	1928–30	52	203	6.2	3.6	50	40.67	50.94	10.10	95–103	0.69
Nephi	5 300	1908–27	48	.....	.....	3.7	..	41.03	50.23	11.61	88–113	0.94
Sevier Bridge Dam	4 980	1922–30	47	.....	.....	4.9	..	41.10	50.48	10.62	93–113	0.69
Piute Dam	5 900	1918–30	48	.....	.....	3.2	50	38.15	47.97	9.65	86–109	0.69
Milford	4 960	1925–27	50	43	10.0	4.3	51	46.98	59.22	10.08	......	0.99
Milford	4 960	1925–27	50	43	10.0	4.3	51	48.53	60.97	10 35	......	0.69
(*q*) Nevada												
Lamoille	....	1918–30	45	.....	.....	2.2	..	27.88	39.45	7.35	87–111	0.69
Fallon	3 970	1908–30	51	.....	.....	3.0	52	44.50	56.74	11.10	80–112	0.94
Lahontan	4 150	1925–30	54	.....	.....	3.3	..	39.83	52.97	9.60	......	0.83
Indian Springs	3 136	1917	..	.....	.....	....	..	51.57	66.06	10.76	......	0.69
Pahrump	2 640	1920–25	62	.....	.....	1.6	..	41.97	56.29	9.24	......	0.69
(*r*) Arizona												
Roosevelt Dam	2 175	1916–30	67	.....	.....	1.4	..	41.80	56.62	10.20	81–115	0.69
Granite Reef	1 310	1910	69	.....	.....	....	..	52.63	79.51	9.84	......	0.69
Mesa	1 225	1917–30	68	107	4.9	1.5	44	37.55	53.54	8.84	86–114	0.69
Willcox	4 190	1917–30	60	.....	.....	3.0	..	40.00	60.23	9.61	82–121	0.69
Tucson	2 400	1929–30	67	.....	6.4	1.3	40	41.82	60.26	9.39	99–101	0.69
Lees Ferry	3 140	1921–30	61	.....	.....	1.9	..	46.09	60.85	10.73	......	0.69
Yuma Reservoir	127	1903	72	.....	.....	....	..	48.05	68.85	9.46	......	0.83
Yuma (Citrus)	220	1920–30	69	.....	.....	2.5	..	58.47	83.38	14.04	92–112	0.69
Yuma (Date Orchard)	127	1917–29	69	54	5.2	1.3	44	36.29	53.45	9.55	91–116	0.69
(*s*) Washington												
Lind	1 350	1926–30	50	.....	.....	4.3	59	39.64	49.76	10.20	92–107	0.85
Walla Walla	1 000	1917–30	53	65	4.9	2.0	58	29.68	36.85	8.64	88–110	0.69
Lake Kachess	2 270	1917–30	44	.....	.....	2.0	..	15.90	20.15	5.07	92–110	0.69
Yakima	1 060	1910	50	.....	.....	....	..	35.07	47.25	7.41	......	0.69
Wind River	1 000	1926–30	48	.....	.....	3.4	..	20.76	26.81	5.92	85–109	0.69
Lake Cushman	760	1924–25	50	.....	.....	....	..	18.96	21.53	4.52	......	0.83

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity			Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches			Percentage range of annual evaporation	Coefficient for pan
				Near-by U. S. Weather Bureau Station: Height, in feet	Near-by U. S. Weather Bureau Station: Velocity, in miles per hour	At the pan, in miles per hour		April–September	Annual	Maximum per month		
(*t*) Oregon												
Warm Springs Reservoir	....	1927–30	47	.....	.....	2.5	..	38.39	54.04	10.42	......	0.69
Burns	4 125	1914–19	44	.....	.....	4.8	59	37.72	49.05	9.57	91–105	0.94
Cold Springs	623	1914–26,'28	52	17	6.6	....	..	27.29	33.74	7.22	74–127	0.69
Hermiston	610	1912–23	52	.....	.....	3.0	..	33.83	41.31	.....	......	0.94
Hermiston	600	1909–13	52	17	7.9	....	..	41.88	51.41	.....	......	0.83
Moro	1 800	1911–19	48	.....	.....	6.8	..	39.53	54.51	10.86	92–108	0.94
Merrill	4 070	1913–19	46	.....	.....	....	..	28.66	37.90	.....	......	0.83
Klamath Falls	4 100	1924–29	48	.....	.....	....	..	24.08	36.71	6.33	......	0.83
Keno	4 090	1904–09	48	.....	.....	....	..	24.82	30.05	6.65	......	0.83
Ady	4 090	1910	48	.....	.....	....	..	35.54	43.26	7.06	......	0.83
Corvallis	235	1922–30	52	.....	.....	1.5	..	21.88	30.68	5.92	91–106	0.69
(*u*) California												
Tahoe	6 230	1916–30	42	.....	.....	3.0	..	22.34	32.09	6.67	84–110	0.88
Lake Elsinore	4 700	1910–18	51	.....	.....	....	..	26.12	35.82	8.22	77–132	0.83
Clear Lake	4 350	1911–13	44	.....	.....	....	..	26.95	35.54	.....	......	0.83
Coppocks	4 035	1916–29	46	.....	.....	....	..	29.38	38.15	8.02	78–120	0.83
Fall River Mills	3 340	1925–30	50	.....	.....	2.3	..	37.35	47.65	9.07	94–108	0.63
Crane Valley Lake	3 300	1912–20	56	.....	.....	....	..	32.20	44.89	8.84	82–110	0.89
Independence	3 766	1909–11	56	28	6.6	....	45	41.15	55.61	8.62	......	0.83
Independence	3 800	1909–11	56	28	6.6	....	45	38.64	55.26	8.00	......	0.83
Cuyamaca Lake	4 640	1913–15	52	.....	.....	....	..	39.26	58.20	9.71	......	0.73
Cuyamaca Lake	4 620	1913–15	52	.....	.....	....	..	38.75	62.25	8.95	94–106	0.88
East Park Reservoir	1 200	1911–29	59	.....	.....	4.0	..	39.48	50.84	9.51	82–117	0.83
Warner Hot Springs	2 780	1913–15	53	.....	.....	....	..	34.64	52.94	8.67	96–108	0.83
San Luis Rey Creek	2 620	1913–15	56	.....	.....	....	..	29.29	45.77	8.67	83–116	0.83
Dodgeland	160	1919–22	61	.....	.....	2.0	..	34.09	42.06	8.65	90–112	0.69
Biggs	94	1913–19	72	.....	.....	3.8	..	41.40	56.09	11.24	91–107	0.94
Oakdale	215	1918–30	60	.....	.....	5.2	..	45.75	56.89	11.76	92–116	0.69
Kingsburg	285	1881–85	63	.....	.....	....	..	28.60	38.38	.....	......	0.83
La Mesa Reservoir	480	1913–15	66	.....	.....	....	..	36.94	55.46	8.01	......	0.83
Sweetwater Reservoir	150	1889–93	62	.....	.....	....	71	32.54	48.55	7.52	......	0.83
Alvarado	3	1924–30	57	.....	.....	4.8	52	26.79	37.58	5.83	91–106	0.69
Chula Vista	9	1918–30	59	.....	.....	3.8	..	26.84	42.26	5.45	89–108	0.69
Indio	–15	1910	73	.....	.....	3.3	33	64.87	89.49	12.26	......	0.75
Brawley	–113	1910	71	.....	.....	4.1	37	55.31	77.66	10.60	......	0.75
Mecca	–189	1910	72	.....	.....	4.7	38	57.41	80.87	11.41	......	0.75
Near Salton Sea	–230	1910	73	.....	.....	4.1	40	66.35	97.10	13.10	......	0.59
(*v*) Canada												
Keetwatin, Ont	....	1913–28	36	175	6.0	....	72	11.24	14.29	3.84	82–117	0.83
Saskatoon, Sask	....	1918–30	..	.....	.....	....	..	23.51	28.89	7.14	84–135	0.84
Edmonton, Alta	....	1918,'21–22	..	.....	.....	....	..	18.65	24.00	5.46	......	0.84
Lake Newell, Alta	....	1919–25	39	.....	.....	2.3	..	26.37	32.32	7.37	......	0.84
Strathmore, Alta	....	1915	..	.....	.....	....	..	22.09	27.94	5.44	......	0.84
Coaldale, Alta	....	1915–21	39	.....	.....	....	..	25.82	32.42	6.82	......	0.84
(*w*) Caribbean Region												
San Juan, Puerto Rico	82	1919–30	78	.....	11.8	....	78	31.34	55.29	6.54	89–107	0.66
Christiansted, Virgin Islands	20	1919–30	79	.....	.....	....	..	28.45	49.88	6.44	91–114	0.69
Gatun, Canal Zone	85	1912–30	80	.....	.....	7.1	84	22.47	48.38	7.03	78–112	0.83
Pedro Miguel, Canal Zone	55	1919–30	79	.....	.....	5.2	83	19.46	42.59	6.14	88–108	0.83
Gamboa, Canal Zone	85	1918–30	79	.....	.....	5.2	..	16.59	36.76	5.66	82–118	0.83
Alhajuela, Canal Zone	120	1918–30	78	.....	.....	1.8	..	18.66	43.04	6.75	92–108	0.83
(*x*) Hawaii												
Maunawili, Oahu	250	1921–30	72	.....	.....	1.9	..	17.23	30.92	3.80	94–108	0.69
Upper Hoaeae, Oahu	705	1920–30	72	.....	.....	1.2	..	25.74	42.58	5.68	93–112	0.69

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity			Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches			Per-cent-age range of annual evapo-ration	Coefficient for pan
				Near-by U. S. Weather Bureau Station		At the pan, in miles per hour						
				Height, in feet	Velo-city, in miles per hour			April-Sep-tember	An-nual	Maxi-mum per month		
(*y*) Egypt and the Sudan												
Alexandria	....	1920–29	68	.....	5.2	....	72	.....	28.76	.....	......	0.50
Giza (Cairo)	....	1920–27	67	.....	5.2	....	69	.....	40.26	.....	......	0.50
Asyut	....	1920–29	71	.....	5.2	....	54	.....	64.71	.....	......	0.50
Wadi Halfa	....	1905–29	76	.....	5.2	....	32	.....	113.60	.....	......	0.50
Merowe	....	1905–29	83	.....	5.6	....	22	.....	120.79	.....	......	0.50
Atbara	....	1905–29	83	.....	5.6	....	29	.....	123.67	.....	......	0.50
Khartoum	....	1905–29	83	.....	5.6	....	31	.....	107.85	.....	......	0.50
Roseires	....	1905–29	79	.....	5.6	....	56	.....	76.21	.....	......	0.50
Malakal	....	1915–29	79	.....	2.8	....	58	.....	64.71	.....	......	0.50
Wau	....	1906–29	79	.....	2.8	....	63	.....	53.21	.....	......	0.50
Mongalla	....	1906–29	79	.....	2.8	....	70	.....	43.14	.....	......	0.50
(*z*) Turkestan												
Khodjent	1 100	1920–29	..	.....	.....	....	..	33.76	42.84	.....	......	0.95
Fergana	1 300	1920–29	..	.....	.....	....	..	29.25	38.05	.....	......	0.95
Andidjan	1 500	1920–29	..	.....	.....	....	..	29.83	37.99	.....	......	0.95
Osh	3 500	1920–29	..	.....	.....	....	..	25.88	33.96	.....	......	0.95

records have been reduced to the common basis of evaporation from reservoir surfaces, and this fact should be considered in studying the figures given in the accompanying tables.

The records, which were taken by pans for which the coefficients had been determined, have been expanded to the full year where necessary. Where the meteorological data were available, the evaporation for the missing periods (usually the winter months) has been either computed or estimated.

The summary of these records, Table 8, is arranged in geographic order, beginning with the northeastern part of the United States. Following the records of the United States are those for outlying possessions and foreign countries (See Table 8(*v*) to (*z*)).

Omitted from the summary are the records at Lawrence, Boston, and Fall River, Mass., and Rochester, N. Y., which were taken from wooden or fiber tanks, for which no coefficients for reduction to reservoir surface evaporation are available (see Table 9). Of these, the best known are the Boston records, taken by the late Desmond FitzGerald, Past-President and Hon. M. Am. Soc. C. E. From 1876 to 1884, these records were taken by observations on small wooden tanks floating in the Chestnut Hill Reservoir, and as wood does not transmit heat readily, it was found that at times the temperature in the tanks varied as much as 10° F from that of the water in the reservoir itself. The same condition persisted when a large wooden tank, 10 ft in diameter and 10 ft deep, was installed in 1884.[22] A summary of these records as observed is given (Table 8), together with others which can not be reduced to reservoir surface evaporation at this time.

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[22] *Transactions,* Am. Soc. C. E., Vol. XV (1886), p. 596.

EFFECT OF FACTORS INFLUENCING EVAPORATION

The principal factors influencing evaporation are so interdependent that it is difficult to segregate the effect of any one. The relative effect of the variation in any one factor can be shown by selecting, from the many available records, pairs in which two factors are practically the same and the third is widely different. The difference in evaporation may then be considered a measure of the effect of varying the third factor. Table 10 shows the relative effect of each factor.

TABLE 9.—SUMMARY OF EVAPORATION RECORDS FOR WHICH NO REDUCTION FACTORS ARE AVAILABLE

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity			Relative humidity (percentage)	Observed Evaporation, in Inches			Percentage range of annual evaporation
				Near-by Weather Bureau Station		At the pan, in miles per hour		April-September	Annual	Maximum per month	
				Height, in feet	Velocity, in miles per hour						
Lawrence, Mass	100	1888–90	45	.....	.....	....	..	.....	29.16	5.20	87–119
Boston, Mass	135	1876–86	49	.....	.....	10.2	72	.....	39.11	.....	....
Fall River, Mass	130	1899–1901	50	.....	.....	....	72	.....	38.71	6.74	96–105
Rochester, N. Y.	620	1891–1925	50	.....	8.5	6.0	72	.....	31.78	6.86	82–123
Charleston, S. C.	25	1905–23	66	.....	10.0	....	79	29.77	47.04	.....	....
Albuquerque, N. Mex	5 000	1900, 03–04	56	.....	8.0	....	..	.....	70.01	12.63	....
Bumping Lake, Wash	3 400	1912	41	.....	.....	....	..	28.23	32.48	8.26	....
Little Bear Valley, Calif	5 200	1896–97	51	.....	4.9	....	..	29.30	36.91	6.60	....
Kingston, Jamaica	100	1924–30	79	.....	7.6	....	79	.....	74.00	9.44	....

VARIATION OF EVAPORATION IN THE UNITED STATES

Most of the records of evaporation were taken in the western part of the United States, comparatively few being available east of the Mississippi River. These records indicate in a general way, however, the variation in the United States.

The area of lowest evaporation is the Great Lakes region, where it ranges from 15 to 20 in. per year. East of the Mississippi River the evaporation increases from 20 in. in Maine to 43 in. at Birmingham, Ala., and then decreases slightly toward the Gulf Coast, with its greater humidity.

West of the Great Lakes the evaporation increases to 40 in. in the Upper Missouri River Basin. Southwest of the Missouri River, it gradually increases to 70 in. in Southwestern Texas and Southeastern New Mexico.

In the Rocky Mountain region, the evaporation depends largely upon temperature, which decreases with an increase in elevation. As an example of this fact, the records in Table 11 are cited.

As few records are available at the higher elevations, it is impossible to estimate the variation in evaporation in the Rocky Mountain region.

In the inter-mountain region, between the Rockies and the Sierra Nevada, the evaporation ranges from 38 in. in Northern Nevada to 60 in. in Southern Nevada and Arizona.

TABLE 10.—RELATIVE EFFECTS OF TEMPERATURE, WIND VELOCITY, AND HUMIDITY ON EVAPORATION

Station	Temperature, in degrees Fahrenheit	Wind at pan, in miles per hour	Relative humidity (percentage)	Evaporation from reservoir surfaces, in inches
(*a*) RELATIVE EFFECT OF TEMPERATURE				
Washington, D. C.	54	2.3	69	34.53
Austin, Tex.	68	2.0	68	45.53
Difference	14	0.3	1	11.00
Denver, Colo.	49	4.5	64	52.15
Laredo, Tex.	73	3.8	64	67.47
Difference	24	0.7	0	15.32
Wagonwheel Gap, Colo.	34	2.0	64	22.32
Wau, Sudan	79	2.8	63	53.21
Difference	45	0.8	1	30.89
(*b*) RELATIVE EFFECT OF WIND VELOCITY				
Columbia, Mo.	54	1.5	71	28.13
Columbia, Mo.	54	2.9	71	35.82
Difference	0	1.4	0	7.69
Silverhill, Ala.	67	1.8	78	38.27
Crowley, La.	68	2.8	77	45.92
Difference	1	1.0	1	7.65
Provo, Utah	49	1.2	54	30.99
Utah Lake Outlet	48	4.2	53	43.19
Difference	1	3.0	1	12.20
Pedro Miguel, Canal Zone	79	5.2	83	45.11
Gatun, Canal Zone	80	7.1	84	51.47
Difference	1	1.9	1	6.36
(*c*) RELATIVE EFFECT OF RELATIVE HUMIDITY				
Beeville, Tex.	70	5.7	81	54.94
Brawley, Calif.	71	4.1	37	77.66
Difference	1	1.6	44	22.72
Chapel Hill, N. C.	61	1.1	69	28.56
Las Cruces, N. Mex.	60	1.9	41	58.46
Difference	1	0.8	28	30.90
Pedro Miguel, Canal Zone	79	5.2	83	42.59
Roseires, Egypt	79	5.6	56	76.21
Difference	0	0.4	27	33.62
Malakal, Sudan	79	2.8	58	64.71
Mongalla, Sudan	79	2.8	70	43.14
Difference	0	0	12	21.57

In the arid section of Washington and Oregon, comprising chiefly the region east of the Coast Range, evaporation ranges from 40 to 50 in. In the western half of Washington, the evaporation is generally less than 25 in., the lowest being at Lake Kachess, with a rate of 20.15 in. In Western Oregon, it is between 30 and 40 in.

In California, the rate of evaporation depends largely upon the elevation of the station. In the Sierra Nevada it ranges from 32 in. at Lake Tahoe to 62 in. in the southern part of the State. Throughout the remainder of California it ranges from 38 to 57 in., except in Imperial Valley, where an extreme of 97.10 in. has been recorded near Salton Sea, in the center of the valley.

TABLE 11.—EFFECT OF TEMPERATURE AND ELEVATION ON EVAPORATION

Station	Elevation, in feet	Temperature, in degrees Fahrenheit	Evaporation, in inches
Wagonwheel Gap, Colo	9 610	34	22.32
Therma, N. Mex	8 200	43	39.17
East portal, Strawberry Tunnel, Utah	7 600	36	19.72
West portal, Strawberry Tunnel, Utah	7 500	41	26.12
Santa Fe, N. Mex	7 010	48	44.82

STANDARD EQUIPMENT FOR EVAPORATION STATIONS

FINAL REPORT OF SUB-COMMITTEE ON EVAPORATION OF THE SPECIAL COMMITTEE ON IRRIGRATION HYDRAULICS

The recommendations of the Special Committee on Irrigation Hydraulics regarding standard equipment at evaporation stations, operated primarily to determine the evaporation from large water surfaces in the immediate neighborhood, are set forth as follows.

PANS

Two or more pans are highly desirable. If only one is feasible, the Class *A* land pan is preferred. If more than one is justified, the order of preference is as follows:

(*a*) United States Weather Bureau Standard Class *A* evaporation (land) pan. (A round land pan; 4 ft in diameter and 10 in. deep; of 22-gauge galvanized iron, without paint; supported on grillage; bottom of pan is 6 in. above ground surface. Details of set-up and procedure may be found in "Instructions for Installation and Operation of Class *A* Evaporation Stations."[23])

(*b*) Modified Colorado "buried" pan. (A square land pan; 3 ft on side; 18 in. depth; sunk in ground to within 4 in. of top; made of 18-gauge galvanized iron, without paint.)

(*c*) United States Geological Survey floating pan. (A square floating pan; same dimensions and material as in Preference (*b*); supported by two cylindrical metal floats, each 9 in. in diameter and 42 in. long; tank floating with 3 in. of its rim above the water surface.)

ESSENTIAL EQUIPMENT

Two items of equipment may be considered essential, namely:

One gauge for measuring evaporation.
One standard Weather Bureau rain gauge.

AUXILIARY EQUIPMENT

The following auxiliary equipment is listed in order of importance:

(*a*) One anemometer for recording wind velocities.

(*b*) One set of maximum-and-minimum thermometers, for air temperatures, housed in the standard instrument shelter of the U. S. Weather Bureau.

(*c*) One set of maximum-and-minimum thermometers for water temperatures.

(*d*) One sling psychrometer, to determine the relative humidity of air.

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[23] *Bulletin* 559, U. S. Weather Bureau, *Circular L*, Instrument Div.

Alternate Equipment.—The following equipment may be substituted under the conditions stated:

(*b′*) Alternate to (*b*); one ordinary thermometer for air temperatures. Where readings can be taken at 12-hr intervals.

(*c′*) Alternate to (*c*); one ordinary thermometer for water temperatures. Where readings can be taken at 12-hr intervals.

(*b″*) Alternate to (*b*) or (*b′*); one thermograph for continuous record of air temperatures.

(*d′*) Alternate to (*d*); one recording hair-hygrometer for continuous record of humidity.

PAN COEFFICIENTS

Greater depth of water is evaporated from pans than from reservoir surfaces; hence, a coefficient is necessary to convert pan evaporation to the equivalent evaporation from a reservoir. Table 12 gives type of pan, recommended coefficient, and observed range of this coefficient.

TABLE 12.—RECOMMENDED COEFFICIENTS AND OBSERVED RANGE

Type of pan	Coefficient	Reasonable range of coefficient
(*a*) Class *A* land pan	0.70	0.60 to 0.82
(*b*) Modified Colorado "buried" pan	0.78	0.75 to 0.86
(*c*) U. S. Geological Survey floating pan	0.80	0.70 to 0.82

REASONS FOR SELECTION OF CLASS *A* LAND PAN AS STANDARD

The first selection must be made as between land pans and floating pans. The Sub-Committee reports two arguments in favor and four against the use of floating pans.

In favor of floating pans are the following:

(1) The amount of evaporation from the pan more nearly approximates that from the surrounding water; that is, a coefficient nearer 1.00 can be used.

(2) In controversial matters, the observations on a floating pan would be more acceptable to many engineers than those from a land pan.

Against a floating pan are the following:

(1) The ever-present possibility of the pan shipping water from the reservoir or losing water due to pitching of waves.

(2) A question as to the effect of measures to prevent shipping or loss of water. The same measure that retards gain or loss of water may set up a condition that increases or diminishes the actual evaporation loss in the pan, compared with that in the reservoir.

(3) Comparative difficulty of access may lead to poor attendance and to "desk readings."

(4) In controversial matters, records are difficult of unquestionable proof.

The items against the floating pan were determined to outweigh those in its favor so the land pan was preferred. The specific reasons are as follows: The U. S. Weather Bureau Standard Class *A* evaporation pan is selected as first choice because it has more features in its favor and fewer unsatisfactory ones than any other type of pan now known or thought obtainable by new design. It does not approach the ideal. Other pans excel it in certain respects, but are far behind it in others.

A summary for and against the U. S. Weather Bureau, Class *A* pan is as follows:

The advantages are:

(1) More data on this pan are available for comparison than on any other particular type of pan.

(2) The coefficient has been found to be about the same for this pan in comparison with large water surfaces in many locations and under many conditions.

(3) It is easy of access for observations.

(4) It is not subject to uncertainty due to inward or outward wash of water as is the case for a floating pan.

(5) It is raised above reasonable drift of dirt, débris, and snow.

(6) It is reasonable in cost of installation.

The disadvantages are:

(1) The coefficient is considerably less than unity; that is, the rate of evaporation is much greater from the pan than from a reservoir surface.

(2) For the data available, coefficients for certain months vary greatly from those for other months. This inconsistency, however, is true for all types of pans thus far studied.

By Sub-Committee on Evaporation,

Fred C. Scobey, *Chairman,*
Ivan E. Houk,
R. L. Parshall,
Carl Rohwer, *Co-Operating Member.*

June 18, 1932.

DISCUSSION

Ralph R. Randell,[24] M. Am. Soc. C. E. (by letter).—Wind, temperature, and humidity are the only factors the variations of which produce effects on reservoir evaporation sufficiently large to be important in most water supply problems. Therefore, if any collection of true reservoir evaporation data is properly adjusted to a given value of one of these three factors, the adjusted data should plot approximately consistently against the other two factors as co-ordinates.

The writer has investigated considerably, although not exhaustively, the question of whether, and to what extent, the annual evaporation data given in Table 8, of Mr. Follansbee's paper, meet this test. The tabulated annual evaporations were adjusted to fixed values of one or another of the three principal influencing factors on the basis of rational and widely held concepts of the effect of the factor upon evaporation. The annual rates of evaporation, as so adjusted, fail to plot even approximately consistently against the other two influencing factors as co-ordinates.

This failure of the adjusted figures to plot consistently raises a doubt in the writer's mind as to the approximate correctness of the data given in Table 8, and of the pan coefficients on which they are partly based.

Possibly, in the light of their intimate familiarity with the work, the authors, or the members of the Committee or Sub-Committee, may be able to devise an adjustment which will give results that plot consistently. It is earnestly hoped that they will attempt to do so, and will report thereon fully. This seems desirable for two purposes: First, to indicate what degree of accuracy and reliability the data possess; and, second, to put the data in such a form that they can be applied and used most intelligently and conveniently.

It is suggested that, first, all the yearly evaporation rates be adjusted to a common basis—as, for example, to a basis of 60% humidity, or a wind velocity of 4 miles per hr—and that the method of making the adjustment be stated. The adjusted evaporation rates should then be plotted against the two other principal evaporation-influencing factors as co-ordinates, the value of each adjusted rate, in inches per annum, being shown at the plotted point on the graph in small figures. Finally, generalized curves representing yearly evaporation rates of 10 in., 20 in., 30 in., etc., should be added.

Such a graph will show what degree of consistency the data possess, and, together with the adjustment formula on which they are based, will give the best obtainable approximation of the annual rate of reservoir evaporation corresponding to any given average values of wind, temperature, and humidity. Even if the given data plot somewhat erratically, the graph, nevertheless, may be of value in suggesting changes—either in the coefficients, the methods of adjustment, or otherwise—which will bring the data into better or more apparent consistency.

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[24] Senior Engr., Federal Power Comm., Washington, D. C.

C. E. Grunsky,[25] Past-President, Am. Soc. C. E. (by letter).—It is with some reluctance that the writer undertakes to contribute his views on the determination of evaporation from large water surfaces to those expressed by the Sub-Committee on Evaporation of the Special Committee on Irrigation Hydraulics in its final report. He feels, nevertheless, that the profession has gone so far astray in its methods of evaporation observation in the past that the data on which the Sub-Committee's conclusions are based, should not be allowed to pass unchallenged, in order that what is dependable information may be segregated from that which is not dependable.

The writer does not agree with some of the conclusions of the Sub-Committee and will give his reasons briefly.

The Sub-Committee states that "if only one [pan] is feasible, the Class *A* land pan is preferred." The writer contends that this is a type of pan that should never have been brought into use, for the reason that its arrangement is such as to give maximum influence upon evaporation of sunshine and air movement. Even if a more or less consistent relation can be found between the record from such a pan and that from a near-by large water surface, it must be remembered that the heat due to sunshine is fed into the water from all parts of the pan that the sun reaches and that, consequently, the evaporation from an open water surface, wherever there is much sunshine, must be much less than the evaporation from the pan. This fact is apparent from the data cited, which indicate that generally such a pan will evaporate nearly 50% more water than a near-by open water surface will lose by evaporation.

By exposure to air on all sides and beneath it, maximum effect is given to air temperature changes. There is more warming and more cooling by contact of the air with the sides and bottom of the pan and with the surface of the water in the pan than if the water surface alone were exposed.

The correct type of standard land pan should preferably be circular, about 4 ft in diameter, with water about 12 to 15 in. deep, sunk into the ground to above the water level, and provided with an inverted V-shaped rider to shade the sides of the pan down to the ground on the outside and down to the water on the inside. The results of observations from a pan of this type will certainly establish relative rates of evaporation in different parts of the country much better than the Class *A* pan with its disturbing, constantly varying, sunshine factor.

The Sub-Committee recommends the use of the U. S. Geological Survey floating pan, but fails to state that no floating-pan record should be regarded as dependable and comparable with other records unless the water in the pan is at all times at a lower level than the water outside the pan. When this is not the case, heat is fed into the water of the pan by the sides, which capture heat from sunshine and air, and, in consequence, the pan record must exceed the actual evaporation from the water in which it floats, instead of closely approximating it. The Sub-Committee at least should add to its floating-pan recommendation the condition that the water within the pan at all times must be lower than the outside water. Undoubtedly, even the late Desmond

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[25] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif. Mr. Grunsky died on June 9, 1934.

Fitzgerald, Past-President and Hon. M. Am. Soc. C. E., failed to recognize the importance of this requirement, with the result that, at times, he found a difference of as much as a 10° F between the temperature of the water in his tanks and that of the surrounding water.

The experimental observations at the Salton Sea, part of which are reported by Mr. Rohwer in Table 3 (*c*), are to be used with extreme caution. The set of pans, for example, that were termed floating pans in the original record, was not of the correct type. They were purposely so suspended in the water that the water within the pans was 3 to 4 in. higher than the surrounding water. This fact was discovered by the writer some years after the experiments had been made, when he found that observation results would not agree with his own ideas as to evaporation rates in that locality. The only effect of the surrounding water at Salton Sea was to influence in some degree the water temperature in the pans. Unfortunately, these misleading records are being constantly quoted because the true circumstances are not known.

It may be assumed that nearly every floating pan of which there is record was improperly suspended. The endeavor has been to keep the top of the pan so far above the surrounding water that water from the outside will not splash in, and no regard has been had to the fact that when the inside water is above the surrounding water, all the heat fed into the sides of the pan above the water line goes into evaporation, which, instead of conforming to that of the outside water, is unduly accelerated. Here, too, therefore, a new start is desirable.

The writer calls attention in this connection to the statement by Mr. Rohwer that the Salton Sea pans referred to in Table 3 (*b*) were well immersed. They were not well immersed, and they should not be called floating pans. Referring to observations at Bakersfield ("General Comparisons: Land Pans and Floating Pans"), Mr. Rohwer decribes the floating pan as having the water on the inside 3 in. above that on the outside. The floating pan at Independence, likewise, is described as having had water at about 2 in. below its rim. The top of the pan was certainly higher than this above the outside water. Here, again, the record has but little value as a floating-pan record. Referring to the records from floating pans taken for the Canadian Pacific Railroad Company, Mr. Rohwer states that the water in the floating pans was kept slightly above the level of submergence. Even at the Fall River Mills, in California (see Table 2(*g*)), the writer some years ago found the water in the floating pan slightly above the outside water. Consequently here, too, there was undoubtedly slightly more evaporation from the pan than from the surrounding water.

As late as 1930, under expert direction, floating pans were placed in Suisun Bay, California, for a determination of evaporation in connection with studies for a salt-water barrier. The writer called attention to the fact that these were insufficiently immersed and that heat from the sides was being fed into the water of the pan, thus accelerating evaporation. At his suggestion, a comparison with a properly arranged floating pan was made. Although this comparison covered only the two cool months of October and

November, nevertheless it showed that the loss from the pan that had its water above the level of the outside water was 10% greater than the loss from a correctly arranged floating pan.

The writer makes the plea that the Sub-Committee revise its report; that the Type *A* pan be condemned; that the U. S. Weather Bureau be urged to make a new start, in the matter of evaporation observations, with land pans sunk in the ground, with the sides shaded, and with floating pans that have the inside water at all times below the outside water. It is only thus that results may be hoped for, which will stand comparison and which will help to establish some usable relation between meteorological conditions and the total evaporation from large open water surfaces in such periods of time as a month or a year.[26]

CHARLES H. LEE,[27] M. AM. SOC. C. E. (by letter).—The Special Committee on Irrigation Hydraulics is to be congratulated upon its accomplishment in producing a study of evaporation which is of practical value to the hydraulic engineer. Its selection of the U. S. Weather Bureau Class *A* land pan as a standard is thoroughly justified by all the facts. The writer concurs in this selection on the basis of his own experience with various types of pans, especially during the five years, 1928 to 1932. The findings of the Committee regarding the coefficient to be applied to the so-called U. S. Geological Survey floating pan are at variance with the view previously held by many, including the writer, that evaporation observed in such a pan closely approximates that from the surrounding reservoir or lake surface. The experimental results, however, would seem to support the coefficient of 0.80 adopted by the Committee.

S. T. HARDING,[28] M AM. SOC. C. E. (by letter).—This Symposium brings together much information that will be helpful to engineers having to do with work in which evaporation from water surfaces may be involved. The presentation of available records obtained from various types of pans and their relationship to the evaporation from large water surfaces furnishes a guide for the interpretation of existing data. The recommendations regarding standard pans for use in future observations will enable many of the variations of past observations to be avoided.

Measurement of evaporation presents many difficulties. This is particularly true of floating pans where, if the pan is placed far enough from the shore to secure conditions of exposure that are representative of the larger water surface, it is difficult to secure freedom from wave splashing without in turn affecting the exposure. Unforeseen conditions may arise which are difficult to overcome. In the writer's floating-pan records on Buena Vista Lake, which are quoted in the Symposium, inconsistencies occurred in the observations for the earlier months which could not be explained until the observer chanced to see pelicans using the pan as a bath-tub. It was necessary to place a coarse wire screen above the pan to prevent this loss of water.

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[26] "Physical Data and Statistics," by William Ham. Hall, M. Am. Soc. C. E., State Engr. of California, 1886, p. 379; *Engineering News*, Vol. 60, August 13, 1908, pp. 163-166; *Bulletin No. 100*, U. S. Dept. of Agriculture, p. 323; *Monthly Weather Review*, 1908 and 1910; and, "Evaporation from Lakes and Reservoirs,' by the late C. E. Grunsky, Past-President, Am. Soc. C. E., *Monthly Weather Review*, Vol. 60, January, 1932.

[27] Cons. Hydr. Engr., San Francisco, Calif.

[28] Prof., Irrig., Univ. of California, Berkeley, Calif.

While the material in the Symposium covers the effect of type and size of pan thoroughly, there are other factors which affect the relationship of the evaporation from pans and from large water surfaces which may make as much difference as variations in the size of pan. This is illustrated by the two following cases.

In describing[29] the work of the U. S. Geological Survey in the Escalante Valley, Utah, Mr. W. N. White gives the results of observations of pans 20 in. and 12 ft in diameter. Two of the smaller pans were used. In one set of observations the two smaller pans floated in the 12-ft pan, being set in the direction of the prevailing wind; in the other set, they were set in the ground adjacent to the 12-ft tank in the same directions. From May to August, 1927, the 12-ft pans had an evaporation of 23.34 in., the windward floating pan, 28.44 in., and the leeward floating pan, 24.49 in. For a period in August and September, 1927, the 12-ft pan had an evaporation of 7.89 in., the small sunken land pan on the windward side, 10.15 in., and the similar land pan on the leeward side, 8.76 in. In the case of the small floating pans the evaporation on the windward pan was 16% greater than that for the leeward pan; for the two sunken land pans, the evaporation on the windward pan exceeded that on the leeward pan by the same average percentage.

In Table 8 of Mr. Follansbee's paper the mean evaporation from April to September from a reservoir surface at Fallon, Nev., is given as 44.50 in., based on records from 1908 to 1930, with a pan having a coefficient of 0.94. This is the pan at the Fallon Experiment Station of the U. S. Department of Agriculture. Observations were begun in 1908 and were continued to 1920, when the station was closed until 1922, when it was re-opened. The evaporation observations have been continued to date (1933). Recently, while making some studies of evaporation in this area, the writer noted that the results since 1922 showed smaller amounts of evaporation than those for the earlier period. For the months from April to September, for the 12 years, from 1908 to 1919, the evaporation from the pan varied from 47.13 to 55.04 in., with a mean of 51.64 in. For the 10 years, from 1923 to 1932, the evaporation for the same months varied from 36.47 to 44.97 in., with a mean of 39.45 in. The mean for the earlier period exceeded that for the latter period by 30%; the evaporation in the maximum year in the later period was only 95% of the evaporation in the minimum year of the earlier period.

Mr. E. W. Knight, in charge of the Experiment Station, states that when the station was re-opened in 1922, the evaporation pan was moved from a small hill in the farmed area of the station to a level area near on irrigated field. Records of wind movement obtained adjacent to the pan show an average reduction in wind velocity of about 25% at the site used since 1922. There has been little difference in mean temperature for these months. While there are no records on large water areas in this vicinity covering both parts of this period, there is no reason to expect that records would have shown any essential differences in actual evaporation from such large water surfaces in the earlier and later years.

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[29] *Water Supply Paper 659-A,* U. S. Geological Survey.

In the observations in Escalante Valley, a water surface 12 ft in diameter set between two small pans resulted in differences in evaporation of 16% due to differences in humidity or other factors. In the observations at Fallon moving the evaporation pan from a small hill about 10 ft to a level area near an irrigated field, but still within the same 160-acre tract and subject to the same general climatic conditions, resulted in a 30% difference in evaporation. These results indicate that the conditions of exposure may be a larger factor in the relationship between the evaporation from pans and large water surfaces, than the factors due to the size of the pan.

L. STANDISH HALL,[30] ASSOC. M. AM. SOC. C. E. (by letter).—A great volume of data has been gathered for this Symposium. In a strict sense, the evaporation from reservoir surfaces is still a matter of question. The coefficients for some of the types of pans in use show a wide range, and although reasonable values have been determined for reducing annual pan evaporation to equivalent reservoir-surface evaporation, little is known of the ratio to be applied in individual months.

The engineers of the East Bay Municipal Utility District (and its predecessor, the East Bay Water Company) have made comparisons of evaporation from different types of pans since 1921. In that year four pans were installed at the San Pablo Reservoir, near Oakland, Calif. All the pans were 3 ft square and 1.5 ft deep, two being Type 2 pans and two being Type 4 pans. One land and one floating pan were located near the dam, and another pair near the outlet tower about 7 000 ft up stream from the dam. The floating pans were anchored near the shore, and the land pans were kept about 10 to 20 ft in elevation above the reservoir level. Rain gauges were maintained near the land pans. The pans were moved at various times according to the elevation of the water surface in the reservoir, but were kept in the same relative positions. Due to an error in attaching the floats to the reservoir pans, the water level in the pans stood between $\frac{3}{4}$ in. and 1 in. higher than the outside water surface.

In 1930, a new evaporation station was installed at this reservoir near the dam. The land station consisted of one Type 2 pan and one Type 3 pan, with anemometer, maximum and minimum air thermometer, and standard rain gauge. This station was located on top of the dam. The reservoir station, consisting of one Type 4 floating pan and a standard rain gauge, was located at the same place as the old floating pan near the dam.

At Upper San Leandro Reservoir, 12 miles southeast of San Pablo Reservoir, a land pan (Type 3) and a floating pan (Type 4) were maintained near the dam from 1925 to 1929. The record for this period is fragmentary. In 1930, a new station was installed near the outlet tower. This station was a duplicate of that placed at San Pablo in the same year.

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[30] Chf. Hydrographer, East Bay Municipal Utility Dist., Oakland, Calif.

At Pardee Reservoir, near Valley Springs, Calif., about 80 miles east of Oakland, an evaporation station was installed in 1930. The equipment was similar to that at San Pablo Reservoir, with the exception that a Class *A* Weather Bureau pan was included with the pans at the land station.

The available data, reduced to monthly averages for each of the aforementioned stations, is included in Table 13.

A comparison of the ratio of evaporation for the various types of pans shows a marked difference between the relative evaporation from land pans and floating pans between the winter and summer months. Even as between two land pans of different types the ratio is not constant throughout the year. Evaporation records from stations other than those at the District's reservoirs, although they show different values of the ratio of evaporation from pans of the several types, do not indicate the marked difference between summer and winter evaporation. In many localities the maintenance of evaporation stations during the winter months is impossible because of freezing temperatures, so that a complete annual comparison is not available.

On the basis of these data, the writer cannot agree with the conclusion of the Sub-Committee that the Class *A* Weather Bureau pan (Type 1) is preferable for an evaporation station "operated primarily to determine the evaporation from large water surfaces in the immediate neighborhood." The test at Fort Collins, Colo., on the 85-ft reservoir, shows that the pan floating in the reservoir followed the reservoir evaporation month by month more closely than the land pans. The Class *A* pan showed the greatest range of coefficient. The evaporation varies with wind, temperature, and humidity, and in a reservoir the temperature of the water may differ greatly from that in a small land pan. With regard to the East Bay District's reservoirs, the results obtained from land pans and water pans can be easily explained. The reservoirs are relatively deep and, in the winter months, the water temperature in the reservoir is greater than in the land pans, resulting in relatively greater evaporation. In the summer months the converse is true, with the result of relatively less evaporation.

Judging from the results at some of the other stations, where comparisons of land pans and water pans are available, similar conditions did not prevail. One difficulty in the comparison of land pans and floating pans is that wind conditions may not be the same at both pans, even if the pans are relatively close to each other. A wide range in evaporation can be seen in the results at San Pablo Reservoir since 1930 (Table 13(*b*)). The land pans on top of the dam are exposed to the full sweep of the prevailing winds. The floating pan is sheltered from the wind by the dam, and has materially less evaporation. Another difficulty in the comparison of floating-pan records is that a large majority of the floating pans are not completely immersed, but have the water level in the pan at a higher level than in the surrounding reservoir. The floating pan near the dam at San Pablo Reservoir was not completely immersed from 1921 to 1929, but after 1930 the water level in a pan at the same location was about $\frac{1}{4}$ in. below the water level in the reservoir. The mean annual pan evaporation for the first period was 47.8 in., and

TABLE 13.—Comparison of Evaporation Pans, East Bay Municipal Utility District, Oakland, California

Item No. (1)	Month (2)	Years of record (3)	Precipitation, in inches (4)	Temperature of air, in degrees Fahrenheit (5)	Velocity of wind, in miles per hour (6)	Total Evaporation for the Month, in Inches				Ratio of Evaporation			
						Type 4 pan (7)	Type 3 pan (8)	Type 3 pan (9)	Type 4 pan (10)	Column (7) to Column (8) (11)	Column (9) to Column (8) (12)	Column (10) to Column (8) (13)	Column (10) to Column (9) (14)
(a) San Pablo Reservoir Near Oakland, California, 1921 to 1929													
Location of pan		...	.....	......	......	Near dam	Near dam	Near tower	Near tower	Near dam	......	......	Near tower
Type of pan		...	.....	......	......	Type 4	Type 3	Type 3	Type 4	Type 4 Type 3	......	......	Type 4 Type 3
Diameter, in feet		...	.....	......	......	3*	3*	3*	3*	......	......	......	......
1	January	7	.....	......	......	0.96	0.77	0.63	0.84	1.25	......	......	1.33
2	February	7	.....	......	......	1.51	1.39	1.21	1.28	1.08	......	......	1.06
3	March	7	.....	......	......	2.62	2.36	2.30	2.46	1.11	......	......	1.07
4	April	8	.....	......	......	3.88	3.32	3.51	3.82	1.17	......	......	1.09
5	May	8	.....	......	......	5.63	5.09	5.50	5.81	1.11	......	......	1.05
6	June	8	.....	......	......	6.37	6.12	6.27	6.59	1.04	......	......	1.05
7	July	8	.....	......	......	6.82	6.79	7.29	7.04	1.01	......	......	0.97
8	August	8	.....	......	......	6.52	6.55	6.45	6.35	1.00	......	......	0.98
9	September	8	.....	......	......	5.67	5.58	5.52	6.09	1.01	......	......	1.10
10	October	8	.....	......	......	3.94	3.54	3.49	3.99	1.11	......	......	1.14
11	November	8	.....	......	......	2.33	1.92	1.83	2.38	1.23	......	......	1.30
12	December	8	.....	......	......	1.56	0.93	0.81	1.37	1.68	......	......	1.69
13	Total	...	.....	......	......	47.81	44.36	44.81	48.03	1.08	......	......	1.07
(b) San Pablo Reservoir, Near Oakland, California, 1930 to 1932													
Type of pan		...	.....	......	......	....	Type 2	Type 3	Type 4	......	Type 3 Type 2	Type 4 Type 2	Type 4 Type 3
Diameter, in feet		...	.....	......	......	....	6	3*	3*	......	......	......	......
14	January	3	4.40	45.3	4.57	....	1.086	1.282	1.181	......	1.18	1.09	0.92
15	February	3	1.96	48.5	3.70	....	1.570	1.856	1.306	......	1.18	0.84	0.70
16	March	3	2.20	52.4	3.50	....	2.967	3.305	2.063	......	1.12	0.69	0.62
17	April	3	1.14	54.5	3.28	....	4.240	4.749	3.412	......	1.12	0.81	0.72
18	May	3	0.78	58.2	3.54	....	5.745	6.327	4.332	......	1.10	0.75	0.68
19	June	3	0.26	61.4	3.86	....	6.976	7.577	5.215	......	1.09	0.75	0.69
20	July	3	0.00	64.2	4.21	....	8.107	8.714	6.162	......	1.08	0.76	0.71
21	August	3	0.00	64.3	4.01	....	7.211	7.944	5.347	......	1.10	0.74	0.67
22	September	3	0.03	63.3	3.62	....	5.385	5.929	4.198	......	1.10	0.78	0.71
23	October	3	0.68	59.3	3.88	....	4.054	4.608	3.424	......	1.14	0.85	0.74
24	November	3	2.19	54.1	4.24	....	2.402	2.832	2.460	......	1.18	1.02	0.87
25	December	3	4.41	44.9	4.57	....	1.155	1.431	1.767	......	1.24	1.53	1.23
26	Total	...	18.05	......	......	....	50.898	56.554	40.867	......	......	......	......
27	Mean	...	.....	55.9	3.92	....	....	....	....	......	1.11	0.80	0.72
(c) Upper San Leandro Reservoir, Near Oakland, California, 1930 to 1932													
28	January	3	5.14	42.5	1.73	....	0.792	0.880	1.183	......	1.11	1.49	1.34
29	February	3	2.53	45.8	1.76	....	1.199	1.297	1.368	......	1.08	1.14	1.05
30	March	3	2.69	51.2	1.72	....	2.237	2.294	2.312	......	1.02	1.03	1.01
31	April	3	0.97	52.9	1.69	....	3.493	3.629	3.943	......	1.04	1.13	1.09
32	May	3	1.14	57.6	2.15	....	4.951	5.113	5.172	......	1.03	1.04	1.01
33	June	3	0.35	60.7	2.26	....	5.870	6.101	6.084	......	1.04	1.04	1.00
34	July	3	0.00†	63.7	2.24	....	7.128	7.396	7.396	......	1.04	1.04	1.00
35	August	3	0.00	63.3	2.14	....	6.114	6.387	6.446	......	1.05	1.06	1.01
36	September	3	0.11	61.8	1.81	....	4.504	4.713	5.162	......	1.05	1.15	1.10
37	October	3	0.86	57.9	1.67	....	3.261	3.441	4.125	......	1.06	1.27	1.20
38	November	3	2.62	51.4	1.55	....	1.736	1.923	2.623	......	1.11	1.51	1.36
39	December	3	5.64	41.7	1.56	....	0.846	0.943	1.777	......	1.11	2.10	1.88
40	Total	...	22.05	......	......	....	42.131	44.117	47.555	......	......	......	......
41	Mean	...	......	54.2	1.86	....	....	....	....	......	1.05	1.13	1.08

* Square. † Trace.

TABLE 13—(*Continued*)

Item No.	Month	Years of record	Precipitation, in inches	Temperature of air, in degrees Fahrenheit	Velocity of wind, in miles per hour	TOTAL EVAPORATION FOR THE MONTH, IN INCHES				RATIO OF EVAPORATION			
						Type 4 pan	Type 3 pan	Type 3 pan	Type 4 pan	Column (7) to Column (8)	Column (9) to Column (8)	Column (10) to Column (8)	Column (10) to Column (9)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
(*d*) PARDEE RESERVOIR, NEAR VALLEY SPRINGS, CALIFORNIA, 1930 TO 1932													
Type of pan.....		...	......	......	......	Type 1	Type 2	Type 3	Type 4	Type 1 / Type 2	Type 3 / Type 2	Type 4 / Type 2	Type 4 / Type 3
Diameter, in feet		...	......	......	......	4	6	3*	3*	......	......	......	......
42..	January...	3	3.65	44.2	1.35	0.674	0.522	0.636	1.452	1.29	1.22	2.78	2.28
43..	February..	3	2.83	48.7	1.10	1.574	0.983	1.173	1.420	1.59	1.18	1.43	1.21
44..	March....	3	1.71	55.1	0.97	3.014	2.124	2.281	2.474	1.42	1.07	1.16	1.08
45..	April......	3	1.13	60.2	0.75	4.756	3.720	3.870	4.319	1.28	1.04	1.16	1.12
46..	May......	3	1.01	66.0	1.12	7.811	6.079	6.323	6.198	1.29	1.04	1.02	0.98
47..	June......	3	0.14	73.1	1.37	10.362	8.078	8.558	7.610	1.28	1.06	0.94	0.89
48..	July......	3	0.00	80.7	1.37	13.076	10.210	10.898	10.215	1.28	1.07	1.00	0.94
49..	August....	3	0.05	78.3	1.20	11.219	9.019	9.625	9.706	1.24	1.07	1.08	1.01
50..	September.	3	0.05	72.1	1.13	7.757	6.077	6.674	7.204	1.28	1.10	1.18	1.08
51..	October...	3	0.39	64.5	1.03	4.887	3.563	4.260	5.241	1.37	1.20	1.47	1.23
52..	November.	3	1.70	57.0	0.95	2.324	1.663	2.108	3.367	1.40	1.27	2.03	1.60
53..	December.	3	3.06	45.6	1.17	0.803	0.575	0.666	2.383	1.40	1.16	4.14	3.57
54..	Total.....	...	15.72	......	......	68.257	52.613	57.072	61.589	......	......	......	......
55..	Mean.....	...	......	62.1	1.13	....	....	....	....	1.30	1.09	1.17	1.08

* Square.

for the second period, 40.9 in. Some of this difference may have been due to climatic variation and some to the inflow of cold water from Pardee Reservoir during the latter period. It does not seem probable that the entire difference was due to pan immersion.

Buried pans may be subject to varying temperature control. When buried in clay or adobe soil, the shrinkage of the earth around the pan during the summer months may result in the sides being exposed to the air. Earth may be removed from around the pan by wind or erosion, exposing the sides below the water level. Pans buried near the water edge of a reservoir or in marshy ground may, in effect, simulate a floating pan. This condition applies to the buried pan at Los Griegos, N. Mex. According to the data[31] originally presented, the ground-water was above the bottom of this pan at all times. Judging from the location of the pans in Fig. 3, a similar condition may have existed relative to the buried pans at the East Park Reservoir, in California.

The Class *A* Weather Bureau pan, being placed above ground, is not subject to the varying temperature control which may effect buried or floating pans. The results, therefore, may be more consistent although the recorded evaporation does not necessarily follow that from an adjacent reservoir. In cases where the pans cannot be given adequate supervision, this pan has many of the advantages set forth by the Sub-Committee, to recommend it.

To the Sub-Committee's recommendation that two or more pans are highly desirable, the writer would add, that if two pans are used, one should be a land pan and one a floating pan. In windy locations, the use of one pan may be misleading. For example, at the San Pablo Reservoir, the writer has used the evaporation of the floating pan since 1930, without correction, as representing most closely the evaporation from the reservoir surface.

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[31] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 984.

Due to the difficulty of maintaining floating pans in a large reservoir, the writer believes that a more accurate comparison of land pans and floating pans could be made by floating the water pan in a tank adjacent to the land pan. For a Type 4 pan, such a tank could be 8 ft in diameter and 3 ft in depth. The tank should be buried in the ground, and an effort made to keep the tank temperature as close as possible to the temperature of the surface water in the reservoir.

The experiments[2] made by the late R. B. Sleight, Assoc. M. Am. Soc. C. E., appear to be very trustworthy. The floating pans, however, cannot be directly compared with the land pans, due to the fact that the pans were not at the same location, as mentioned by Mr. Rohwer. The writer, by comparison of results between land pans and floating pans at Denver, Colo., with the results obtained at Salt Creek Bridge, in California, has revised the coefficients for floating pans obtained by Sleight, as shown in Table 14.

TABLE 14.—Evaporation from Circular Floating Pans (Type 5), and from Square Floating Pans (Type 4).

Type	Diameter, in feet	Area, in square feet	Salt Creek Bridge, California	Denver, Colo., original*	Denver, Colo., revised
5....................	12.0	113.09	1.00	....	....
5....................	6.0	28.27	0.95	1.03	0.93
5....................	4.0	12.56	0.82	0.97	0.87
5....................	2.0	3.14	0.81	0.89	0.80
5....................	0.83	0.54	....	0.77	0.69
4....................	3.0†	9.00	....	0.89	0.80

* From Table 4; from comparison with 12-ft. circular buried land pan not at same location as floating pans.

† Square.

In other instances, in addition to the one just cited, the writer could not agree with the values of the coefficients determined in various parts of the Symposium. The coefficients for the types of pans in most general use, as well as the mean for each type have been assembled in Table 16, and in Table 15 these results are compared with the coefficients determined by

TABLE 15.—Comparison of Coefficients to Reduce Pan Evaporation to Equivalent Reservoir Evaporation

Type	Diameter, in feet	Sleight	Rohwer	Follansbee	Sub-Committee	Hall
1..........	4.0	0.70	0.70§	0.69	0.70	0.69
2..........	6.0	0.92†	0.94	0.94	....	0.91
3..........	3.0*	0.79	0.78	....	0.78	0.81
4..........	3.0*	0.80‡	0.77	0.83	0.80	0.80
5..........	4.0	0.87‡	0.78	0.83	....	0.88

* Square. †Record for pan 3.0 ft. deep used. ‡ Revised coefficient by Hall. § Recommends 0.69 or 0.70.

Messrs. Sleight, Rohwer, Follansbee, and the Sub-Committee. The coefficient of 0.92 for the 6-ft ground pan was determined from a longer record on the pan 3 ft deep at Denver, and is believed to be more representative than the coefficient of 0.94 from the short record on the pan of standard depth.

TABLE 16.—Coefficients to Reduce Various Types of Pan Records to Reservoir Surface Evaporation

Item No.	Location	Reservoir with which pans were compared	Years of record	Class *A* pan (diameter, 4 ft)	COEFFICIENTS				
					Buried Pans			Floating Pans	
					(Diameter, 6 ft)	(Diameter, 4 ft)	3 ft square	3 ft square	(Diameter, 4 ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Denver, Colo.	12-ft ground pan; Column (7) interpolated	1915–1917	0.70	0.92	0.85	0.79	0.80*	0.87†
2	Fort Collins, Colo.	85-ft reservoir	1926–1928	0.70	......	......	0.79	0.77	......
3	East Park, Calif.	1 800-acre reservoir; Columns (7) and (8) (pans semi-floating, not used in averages)	1930	0.68	......	0.78	0.75	......	0.78
4	Milford, Utah	12-ft ground pan	1925–1927	0.67	......	......	......	......	......
5	Salton Sea, California.	175 600 acres; indirect method	1909–1910 1909–1910	0.68 0.66	....	....	....	....	0.85‡ ......
6	Lincoln, Nebr.	7.5-ft pan; indirect method	1917–1920	0.68	......	......	......	......	......
7	Pardee Reservoir, California	4-ft, Class *A* pan	1930–1933	......	0.90§	......	0.84§	0.76‖	......
8	Garnett, Colo.	4-ft, Class *A* pan; indirect method	1927–1930	......	......	0.88	......	......	......
9	Bakersfield, Calif.	10 000-acre lake; Column (9), pan not fully submerged	1920	......	......	......	0.82	0.70	......
10	San Pablo Reservoir, California	6-ft ground pan	1930–1933	......	......	......	0.82§	0.75¶	......
11	Upper San Leandro Reservoir, California	6-ft ground pan; indirect method	1930–1933	......	......	......	0.87	0.75‖	......
12	Austin, Tex.	4-ft, Class *A* pan; indirect method	1916–1925	......	......	......	......	0.88	......
13	Morena Reservoir, California	3-ft square ground pan; indirect method	1928	......	......	......	......	0.92	......
14	Barrett Reservoir, California	3-ft square ground pan; indirect method	1928	......	......	......	......	0.86	......
15	Lower Otay Reservoir, California	3-ft square ground pan; indirect method	1928	......	......	......	......	0.84	......
16	Los Griegos, N. Mex.	4-ft, Class *A* pan in saturated soil; indirect method	1926–1928	......	......	......	......	......	0.97
17	Nelson Reservoir, Montana	4-ft, Class *A* pan; not used in average; indirect method	1921–1923	......	......	......	......	......	1.01
18	Oakdale, Calif.	4-ft, Class *A* pan; not used in average; indirect method	1921	......	......	......	......	......	1.08
19	Fall River Mills, Calif.	4-ft, Class *A* pan; indirect method	1925–1930	......	......	......	......	......	0.91
20	Salt Creek Bridge, California	12-ft circular floating pan	1910	......	......	......	......	......	0.83
21	Brooks Reservoir, Alberta, Canada	4-ft circular ground pan; not used in average; indirect method	1923–1924	......	......	......	......	......	1.09
22	Lake Newell, Alberta, Canada	4-ft circular ground pan; indirect method	1923–1924	......	......	......	......	......	0.98
23	Means	......	......	0.69	0.91	0.86	0.81	0.80	0.88

* 1916 to 1917; indirect method. † 1917; indirect method. ‡ 1910. § Indirect method. ‖ 1930 to 1932. ¶ 1921 to 1929; average of two pans; indirect method.

The comparisons of the Type 3 and Type 4 pans indicate that the evaporation is greater from the floating pan. The results from the East Bay District's measurements are very consistent and show 8% greater evaporation from the floating pan. Based on these records alone, the coefficients would be: Type 3, 0.85; and Type 4, 0.79. The coefficient for the Type 5 pan appears to be 0.88.

In selecting a pan to determine reservoir evaporation, the writer would reverse the selection recommended by the Sub-Committee and would give preference to the pans in the following order: (*a*) Floating pan, with preference to the U. S. Geological Survey pan; (*b*) buried pan, with preference to the Colorado pan, and then to the 4-ft or 6-ft circular pan; and (*c*) U. S. Weather Bureau Class *A* pan.

ADOLPHO SANTOS, JR.,[32] JUN. AM. SOC. C. E. (by letter).—The primary object of the Symposium "has been to find a suitable set of coefficients * * * to the end that evaporation from a large water surface could be expressed in terms of that from the various types of pans in use." In the light of certain researches and experiments conducted on the subject of evaporation from large water surfaces, it seems that the appropriateness of the method of attack chosen for obtaining such a suitable set of coefficients is not beyond criticism.

The researches and experiments referred to, made principally at the Scripps Institution of Oceanography and at the California Institute of Technology, would appear to indicate that the method adopted by the authors for estimating evaporation from reservoirs on the basis of pan observations yields results of debatable precision. For a proper solution of the problem, the adoption of the point of view and technique originated and advocated by Messrs. G. F. McEwen, N. W. Cummings, I. S. Bowen, B. Richardson, and C. Montgomery, is essential. It is to be regretted that not even passing mention is made by the authors of the work of these men[33] who have succeeded in placing the physics of evaporation on a firm foundation.

Messrs. Cummings and Richardson have stated[34] that:

> "Evaporation can be determined by the aid of the first law of thermodynamics in such a way that wind velocity need not enter the calculation. Air temperature and humidity enter only as terms in a correction which can have a relatively small average value under typical conditions."

As long as this fact is not fully realized and made use of by engineers, and as long as pans of various types and sizes, under varying conditions of exposure, continue to be used, the ratio between evaporation from pans and evaporation from a large open water surface will remain a matter of controversy and dispute, with small gain in factual knowledge of general applicability. As long as empirical methods are used, the "laws," "constants," and "coefficients" resulting from even elaborate experiments must of necessity remain of local and restricted value only.

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[32] Asst. Hydr. Engr., The São Paulo Tramway, Light & Power Co., Ltd., São Paulo, Brazil.

[33] "The Ratio of Heat Losses by Conduction and Evaporation from Any Water Surface," by I. S. Bowen, *Physical Review*, Vol 27 (1926), pp. 779–787; "The Measurement of Insolation by Means of a Pan," by B. Richardson and C. Montgomery, *Bulletin No. 68*, National Research Council (1929), pp. 56–61; "A Mathematical Theory of the Vertical Distribution of Temperature and Salinity in Water Under the Action of Radiation, Conduction, Evaporation, and Mixing Due to the Resulting Convection," by G. F. McEwen, *Bulletin*, Scripps Inst. of Oceanography, Technical Ser., Vol. 2, No. 6 (1929), pp. 197–306; "Alignment Diagram for 'R' of the Energy-Evaporation Equation," by N. W. Cummings, *Monthly Weather Review*, Vol. 58 (1930), pp. 142–144; "Certain Limitations on the Possible Values of the Ratio of Heat Losses by Convection and by Evaporation at a Water Surface," by N. W. Cummings, *Monthly Weather Review*, Vol. 58 (1930), pp. 144–146; "Results of Evaporation Studies Conducted at the Scripps Institution of Oceanography and the California Institute of Technology," by G. F. McEwen, *Bulletin*, Scripps Inst. of Oceanography, Technical Ser., Vol. 2, No. 11 (1930), pp. 401–415, and "Evaporation as a Function of Insolation," by B. Richardson, *Transactions*, Am. Soc. C. E., Vol. 95 (1931), pp. 996 *et seq.*

[34] "Evaporation from Lakes", by N. W. Cummings and B. Richardson, *Physical Review*, Vol. 30 (1927), pp. 527–534.

For example, in water supply problems, it is generally held that wind velocity, temperature, and humidity are the only factors that produce effects on reservoir evaporation sufficiently large to be important. It happens, however, that this very natural and common view was not supported by an extensive statistical analysis made by Dr. Cummings for the purpose of isolating the effects of the various factors influencing evaporation. In fact, a very weak correlation was indicated between evaporation and humidity and between evaporation and wind.[35] This led Dr. Cummings to attack the problem from the standpoint of the first law of thermodynamics and to formulate the fundamental qualitative principle of evaporation, as follows:

> "Any change in wind or humidity is always accompanied by a change in lake temperature which partially neutralizes the direct effect on the evaporation rate to be expected from the original change in atmospheric conditions, if the insolation remains constant."

It would appear, therefore, that if one should adjust evaporation rates, by means of the empirical formulas in common use, to fixed values of one or another of the three factors—wind, temperature, and humidity—it will be found that after adjustment the data do not plot consistently against the other two factors as co-ordinates.

When the water in a floating pan is kept above the level of submergence, the rate of evaporation from the pan will exceed that of the surrounding water. The writer is not certain, however, that by simply keeping the water in the pan at all times at a lower level than that of the water in which it floats the two rates will become identical. The first question that naturally arises is: How much lower? Is it not conceivable that if the level is low enough the evaporation from the pan will be less than that from the reservoir?

The writer earnestly hopes that the Sub-Committee will adopt the standard procedure and technique advanced by Dr. McEwen, and his co-workers. Like most primary standards it is difficult to apply. By this means, however, a simpler and more readily applicable method could be developed for determining evaporation from reservoirs with the degree of accuracy sufficient for engineering uses.

The words of Dr. Cummings in discussing Professor Richardson's paper,[36] with slight modification, may be applied to the question being considered:

> "It may be some time before the true value [of the procedure based on the first law of thermodynamics] is fully appreciated, because economic pressure has not yet compelled the abandonment of unnecessary complications in estimating evaporation from lakes and reservoirs."

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[35] "Relation Between Evaporation and Humidity", by N. W. Cummings, *Bulletin No. 68*, National Research Council (1929), pp. 47–56.

[36] *Transactions*, Am. Soc. C. E., Vol 95 (1931), p. 1017.

John W. Pritchett,[37] Assoc. M. Am. Soc. C. E. (by letter).—While this Symposium brings together many evaporation data not heretofore collected in one publication, it is also noted that many evaporation records in Texas, covering long continuous periods, are not disclosed. This is doubtless true of evaporation data covering other sections of the United States. There are many evaporation records that have never been published, which would be of much value to the profession. The U. S. Weather Bureau would be a good agency to handle such work if allowed the necessary money.

Evaporation records beginning in 1915 or 1916 are being taken at eleven Texas Agricultural Experiment Stations. Records taken by the Bureau of Agricultural Engineering, in co-operation with the Texas Board of Water Engineers, at Mercedes, in the Lower Rio Grande Valley, are valuable in that comparison may be made of the evaporation from pans of different sizes and under varying climatological conditions for complete yearly periods. The land tanks were 1, 2, and 3 ft in diameter, and the floating tank was 2 ft in diameter.

Many of the Texas data are all-year records and on that account would be particularly valuable in carrying on evaporation studies. The San Antonio Field Station, Bureau of Plant Industry, U. S. Department of Agriculture, has an evaporation and temperature record beginning January, 1907. This is a complete 26-yr. record for a land pan (Type 2), 6 ft in diameter.

Table 5(*b*) shows a comparison of Pan (1) and Pan (4) at Austin, Tex. This is known as the Hill's Ranch Station, and is about five miles southeast of Austin. It was established by the Water Resources Branch of the U. S. Geological Survey in co-operation with the State Board of Water Engineers and with the U. S. Weather Bureau. The records were taken under the direction of the District Engineers of the U. S. Geological Survey, and the computations were made by the Weather Bureau.[38]

This station was established on March 9, 1916. It is 475 ft above sea level, at Latitude 30° 14′ N, and is located about 1 000 ft from the ranch house. Observations are taken daily at 8:00 A. M., 90th Meridian time.

According to the U. S. Geological Survey,[39] the topography is rolling to the north and east and rough to the west and south, with a gradual slope to the Colorado River. In the immediate vicinity of the pan the slope is approximately 30 ft per mile. The reservoir is 30 by 200 ft, instead of 20 by 150 ft, as reported in the Appendix to Mr. Rohwer's paper, and the galvanized iron floating pan (36 in. square and 13 in. deep) is located 30 ft east of the land pan. It is freely exposed to the rays of the sun at all daylight hours.

This is probably one of the most complete records that may be found from which to determine the relationship between the evaporation from pans of Type (1) and Type (4). While the published records frequently show days missing or estimated records, the original records are practically complete. In cold weather when the pans were frozen over, the record was omitted in the publication and noted as days missing. The 14-yr. monthly record similar to Table 5, of Mr. Rohwer's paper, was tabulated by the writer, using all records, and using 32° F for the water surface temperatures when the records showed frozen water surface.

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[37] Engr., State Board of Water Engrs., Austin, Tex.

[38] "Climatological Data, Texas Section," U. S. Weather Bureau, December, 1916.

[39] *Water Supply Paper No. 438*, U. S. Geological Survey, p. 31.

Comparing these results (not published) the writer has found that there is remarkably little difference between them and the data in Table 5. This would seem to indicate that a few years of evaporation record at a given station may suffice for all practical purposes.

The writer agrees with Mr. Rohwer and with the Sub-Committee that from the standpoint of making comparisons with known data on evaporation, the Class *A* Land Pan of the U. S. Weather Bureau (Pan (1)), is probably best adapted to the study of evaporation from water surfaces, because a large number of these pans have been installed in the United States, and many observational data obtained under similar conditions with a uniform procedure, are available from them.

In Table 10, Mr. Follansbee attempts to show the effects of factors influencing evaporation. In Table 10(*a*), showing the effect of temperature, the Washington, D. C., data are compared with the Austin, Tex., data. The 14-yr. record of the Austin Station shows a mean temperature of 67° F compared to 68° F, as shown in the table. The wind velocity at the pan, in miles per hour, is 2.1 compared to 2.0, as shown in the table. The relative humidity is 66 compared to 68. The evaporation for the 14-year record is 45.6 for both Pan (1) and Pan (4), applying a factor of 0.69 to the Pan (1) records and a factor of 0.90 to those of Pan (4).

In comparing stations of approximately the same mean annual temperature, in order to show the effect of wind velocity and of relative humidity on evaporation, it appears to the writer that it would be necessary to know that the stations had not only the same mean annual temperature, but also the same range of temperature.

In establishing the coefficients to be applied to the different types of evaporation pans, in order to reduce the evaporation to the open-water equivalent, it is suggested that a more accurate determination might be made by finding coefficients that would apply to certain temperatures or ranges of temperature. Determining coefficients for individual months would have somewhat the same effect.

Various formulas have been derived from which evaporation estimates may be made that are sufficiently accurate for many practical purposes. The application of such formulas may soon be found to be a more practical manner of estimating evaporation, rather than basing such formulas on evaporation-pan records. Evaporation observations would then only be necessary for scientific purposes, or where extreme accuracy could be attained in obtaining climatological data.

IVAN E. HOUK,[40] M. AM. SOC. C. E. (by letter).—The papers by Messrs. Rohwer and Follansbee furnish a long needed digest of available evaporation data. Mr. Rohwer's paper brings together valuable information regarding comparative evaporation rates observed at different types of pans, and their relations to actual evaporation losses from large reservoir surfaces. Mr. Follansbee's paper presents comprehensive and valuable compilations of pan evaporation data reduced to equivalent reservoir evaporation.

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[40] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

As regards the ratios of pan evaporation to reservoir evaporation, it is not surprising that there should be some differences between results obtained at different locations. Considering the varying meteorological conditions throughout the country it would not be logical to assume that the ratio of the evaporation rate at any given pan to the evaporation loss from a large reservoir surface in the vicinity of the pan should have the same value in all localities. However, considerable porportions of the differences shown in Mr. Rohwer's tabulations are probably due to incidental errors of observations and lack of strictly comparable environment at the different places where the investigations were conducted.

The writer would not expect the data on pan ratios secured at the Stonyford, Calif., installations (Fig. 3), to be susceptible of general application, because of the proximity of the pans to each other and to the shore of the reservoir. The slightest offshore or onshore breeze must have produced essentially the same vapor pressure in the air immediately over the different pan surfaces; so that the only factor causing differences in evaporation rates at the different pans must have been the temperature of the water within the pans. Moreover, ratios of reservoir evaporation to pan evaporation at Stonyford must have been affected materially by the prevailing wind direction. In such an installation prevailing offshore breezes would mean definitely lower ratios of reservoir evaporation to pan evaporation than would be the case if onshore breezes predominated.

In the Stonyford installation the writer would expect the evaporation rate at the floating pan to be essentially the same as at the sunken land pans. Somewhat different ratios of reservoir evaporation to pan evaporation undoubtedly would have been obtained at the different pans if the floating pan had been in the center of the lake and the land pans entirely away from the reservoir. In other words the grouping of the pans near the shore of the lake tended to eliminate one of the variable factors that affect the rate of evaporation. Consequently, engineers cannot apply, blindly, ratios determined at such installations to land pan measurements made in a desert country, entirely away from any large body of water. Such ratios can only be used in the case of pan measurements near large lakes or reservoirs, where the relative humidity above the pan is substantially the same as over the larger water areas.

Mr. Follansbee's paper supplies a wealth of information regarding estimated reservoir evaporation based on pan measurements in different sections of the United States. His tabulations can be used in preliminary investigations of proposed storage projects, selecting the stations most comparable as regards elevation, temperature, relative humidity, and wind velocity. The writer believes that Mr. Follansbee's calculated estimates of reservoir evaporation in Western United States are on the safe side in most cases;

that is, that they tend to be too high rather than too low, if in error at all. The reason for this belief is that in a few cases the pan measurements may have been made in regions where the relative humidity is definitely lower than that at the locations where the pan ratios were determined, and also definitely lower than the relative humidity that will occur in the vicinity of the pan after a large reservoir is constructed.

The Yuma (Citrus) Station, in Arizona, (Table 8 (*r*)) is an example of the aforementioned conditions. In this case the writer believes that the coefficient of 0.69 used to reduce Class *A* pan evaporation to reservoir evaporation is entirely too high. If a large reservoir was built at the location of the Yuma (Citrus) Station the relative humidity of the air above the reservoir surface probably would not be materially different from the relative humidity of the air above a large reservoir at the Yuma (Date Orchard) Station. Consequently, since the wind velocities and air temperatures are practically the same at the two stations, the evaporation rates at the two reservoirs would be essentially the same and the ratio of reservoir evaporation to past pan records at the Yuma (Citrus) Station would be much lower than 0.69. Meteorological conditions at the Yuma (Citrus) and the Yuma (Date Orchard) Stations and their effects on measured pan evaporation were discussed in a previous paper by the writer.[41]

In finally planning any important storage development, the engineer should consider carefully the meteorological conditions at the proposed reservoir site before and after construction and their relations to the meteorological conditions at the places where the pan ratios were determined, before adopting a definite value for the ratio of reservoir evaporation to pan evaporation. The engineer might also, profitably, consult Mr. Rohwer's excellent and comprehensive *Bulletin*[42] which gives the results of recent investigations by the U. S. Department of Agriculture at Fort Collins, Nebr., Logan, Utah, Imperial and Lake Tahoe, California, and Fort Collins, Victor, and Pike's Peak, Colo. In that *Bulletin* Mr. Rohwer not only showed conclusively that the Dalton law, proposed in 1802, constitutes a satisfactory theoretical basis for a practicable evaporation formula, but also developed such a formula from his extensive evaporation measurements and tested it in many different locations and at many different altitudes throughout the country.

In order to use the Rohwer formula in a given case it is necessary to know the altitude, wind velocity, relative humidity, and water surface temperature. Since the altitude is practically always known with a sufficient degree of accuracy, only three factors remain to be determined. Of these three, wind velocity has already been measured more or less extensively at meteorological stations maintained by the U. S. Weather Bureau; so that the principal additional data needed are values of relative humidity and water surface temperature in different sections of the country, both of which quantities are easily measured with relatively inexpensive equipment. Engineers interested in securing better evaporation data should encourage the measurement of relative humidity and water surface temperatures at lakes and reservoirs whenever possible, as well as the establishment of additional evaporation stations and the measurement of all pertinent meteorological conditions at such installations.

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[41] "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 282–285.

[42] "Evaporation from Free Water Surfaces," by Carl Rohwer, Assoc. M. Am. Soc. C. E., *Technical Bulletin No. 271*, U. S. Dept. of Agriculture, Washington D. C., December, 1931.

R. I. MEEKER,[43] M. AM. SOC. C. E. (by letter).—Evaporation losses from reservoirs and river channels, in the Western United States, have become a matter of considerable importance to irrigation engineers, in recent years, due to interstate river problems and numerous proposed channel reservoirs of large capacity. Information and engineering data on evaporation from water surfaces were relatively meager before 1924.

In 1927, Ivan E. Houk, M. Am. Soc. C. E., brought together in compact form valuable information[44] now pertinent to engineering studies of river equalization. The Symposium on "Evaporation from Water Surfaces" is a fitting conclusion to the interest awakened in the profession by Mr. Houk's paper.

The contributions of Mr. Rohwer and of Mr. Follansbee represent extensive research, exacting compilation, and careful analytical study of an immense mass of records—State, regional, continental, and foreign. Table 8 in Mr. Follansbee's paper is a most valuable reference work because of its comprehensive character. Table 10 is most instructive, and forces one to study comparative values of evaporation from the factors presented.

The conclusion of the Sub-Committee, in favor of the Class *A* land pan as to desirability for evaporation records is sustained by the experience of the writer. No floating-pan records secured by him were ever satisfactory as to accuracy, due to indeterminate water gains or losses from wave action.

These papers on evaporation, supplemented by "Evaporation from Free Water Surfaces"[45], by Mr. Rohwer, will furnish a satisfactory library for years to come on that important subject.

CARL ROHWER,[46] ASSOC. M. AM. SOC. C. E., (by letter).—In preparing this paper the writer had no intention to minimize the importance of the work being done by Messrs. Richardson, Cummings, and McEwen on the relation between insolation and evaporation. It is generally conceded that, over a period of time, the quantity of water evaporated will depend on the insolation. There are several practical difficulties, however, which must be overcome before this method can be used for the determination of the evaporation losses from water surfaces and, until more information is available, the method cannot be used generally in evaluating the evaporation loss from a reservoir.

The mere fact that the evaporation data do not plot consistently when adjusted to a fixed value of one of the factors affecting evaporation, does not necessarily mean that the original data, or that the pan coefficients, are incorrect, as suggested by Mr. Randell. The apparent inconsistency in the data may be due to several causes: First, the method of adjusting them to a fixed value of one of the factors affecting the result may be in error; second, the meteorological data given may not have been taken at the evaporation pan; and, third, the meteorological data, although taken at the evaporation pan, may have been observed at such times that the means do not represent the true conditions. Any one or all three factors might cause inconsistencies in the plotted results.

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[43] Cons. Engr., Denver, Colo.

[44] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 266.

[45] *Technical Bulletin No. 271*, U. S. Dept. of Agriculture, Washington, D. C.

[46] Associate Irrig. Engr., Colorado Experiment Station, Fort Collins, Colo.

The writer agrees with the statements of Mr. Grunsky concerning the evaporation from the U. S. Weather Bureau, Class *A* land pan. He admits also that the evaporation from a sunken circular pan provided with a shield to protect the exposed rim from sunshine would approach more nearly the evaporation from a large body of water. Records from the latter type of pan are not available, however. Although the evaporation from the Weather Bureau pan is much greater than that from a lake or reservoir, this type of pan has many advantages, as has been stated, and for these reasons it is recommended for use.

There is a noticeable lack of uniformity in the method of maintaining floating pans, which is, in part, the cause of the inconsistent results obtained from these pans. The data as to the setting of the floating pan for which evaporation records were reported by the writer, were given to acquaint the reader with the conditions under which the readings were taken and not with the intention of recommending the particular setting. As pointed out by the writer, the water in the U. S. Geological Survey floating pan should be maintained at the same level inside as outside.

Mr. Pritchett calls attention to additional records on evaporation from pans of different sizes, and states that the reservoir at Austin, Tex., is 30 by 200 ft, instead of 20 by 150 ft. It is the writer's understanding that the size of the reservoir varies with the rainfall and is at times even smaller than 20 by 150 ft.

The evaporation records submitted by Mr. Hall provide additional information as to the relation between the evaporation from floating pans and from other types. It is unfortunate that this information was not included in the writer's paper, as only a few records concerning the comparison between the evaporation from floating pans and land pans were obtained. The writer does not agree, however, with Mr. Hall's recommendations as to the types of pans to be used for the study of the evaporation from reservoirs. His own records (Table 13) show that the results obtained from the floating pans are more erratic than those from land pans.

The observations by the writer on the floating pan in the 85-ft reservoir at Fort Collins, Colo., were made under laboratory conditions where it was possible to measure the evaporation accurately and to observe whether water was splashing into or out of the pan. It is doubtful whether as consistent results would be obtained under field conditions. The land pans at Stonyford, Calif., were installed in damp sand, but the conditions were scarcely similar to those under which floating pans are exposed.

The ratios given by the writer in Table 6 are based on the available records, but they are not final. Whenever additional information on the evaporation from different types of pans becomes available the ratios given should be recomputed on the basis of all the data.

Mr. Houk calls attention to the effect of variation in the humidity near large bodies of water on the evaporation from pans. It is generally accepted that the air is damper near lakes and reservoirs; but the effect of the water on the evaporation is not as great as might be expected, for two reasons: (1) Because moist air is lighter than dry air, the moist air is rising continuously and being replaced by dryer air (but for this fact and the influence of the wind, the air near a lake or reservoir would soon become saturated); and (2) the rate of evaporation is proportional to the difference in vapor pressure between the water surface and the air above it. In arid sections (particularly in the summer), the vapor pressure of the water surface is much greater than the vapor pressure of the air, and, consequently, small changes in the vapor pressure of the air due to proximity to a large body of water have a relatively small effect on the rate of evaporation of pans near the lake or reservoir. This conclusion is supported by the observations in Table 6, Items Nos. 44 to 49, inclusive, which show that the correction factors for obtaining the reservoir equivalent are practically the same whether determined near a lake or away from the influence of a large body of water.

The observations reported by Professor Harding pertaining to the evaporation from small land and floating pans located on the leeward and windward sides of the 12-ft evaporation pan at Escalante, Utah, indicate that the evaporation is less on the lee side of the large pan. This may be due to the difference in humidity, but other factors, such as wind and temperature, may have some effect also. The effect of wind is clearly shown by Professor Harding by the observations on the pan at the Fallon Experiment Station. Wind is more likely to be the cause of differences in evaporation records than temperature or humidity. For this reason the pan should be exposed as nearly as possible to the same wind conditions as the reservoir for which the rate of evaporation is desired.

In conclusion, the writer would like to call attention to the fact that the ratios given in his paper are averages, usually covering several months, and are based on the total evaporation for the period and not on the average of the monthly ratios. For this reason the ratio for a particular month of the year may differ from the average for the period. This fact should be given consideration when computing monthly rates of evaporation, particularly for the winter months when the pan may be frozen while the reservoir still remains open.

Robert Follansbee,[47] M. Am. Soc. C. E. (by letter).—In publishing all available records of evaporation (the real purpose of the writer's paper) two courses were open: (1) To present the records as observed with the knowledge that, obviously, they were not comparable, owing to the various methods by which they were obtained, and thus could not give even an approximately true picture of the variation in evaporation due to variations in meteorological factors; or (2) with the aid of known comparisons to reduce all records to approximate reservoir surface evaporation which would be much more nearly comparable than the observed records and, at the same time, would present the factor used in each case, in order that the user of the records might easily deduce the originals and thus correct them by any factor he chose, based on additional information. The latter method was selected.

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[47] Dist. Engr., U. S. Geological Survey, Denver, Colo.

In selecting the reservoir surface method the writer was well aware of the fact that in some instances the known reduction factors were probably subject to error, and that the resulting values would appear inconsistent when compared with the accompanying meteorological factors. One reason for these inconsistencies undoubtedly lies in the fact that these factors are not in every instance directly comparable. While an attempt was made to obtain exact information regarding them and the points at which they were observed, such attempt was not always successful. Particularly is this true of the foreign records presented in this closure. One of the greatest uncertainties is the mean value used for relative humidity. At only a very few stations are continuous records of relative humidity available, and the great majority rely on two or three readings per day, taken during the daylight hours. In the case of the Australian records (Table 17 (*g*)) one reading per day is taken. In some climates the error in computing the mean daily relative humidity from one, two, or (at regular U. S. Weather Bureau stations), three, readings per day may give the true daily mean within a few per cent., but in other climates considerable error may be introduced. Mr. Pritchett's discussion of the writer's comparison between the evaporation at Washington, D. C., and Austin, Tex., as given in Table 10, illustrates the variation in relative humidity at different hours of the day. Mr. Pritchett states that the relative humidity at Austin, as shown by a 14-yr record, is 82%, whereas the writer had given 68% in his record. From subsequent correspondence, it has been learned that the value of 82% was based on one reading per day at 8:00 A. M., whereas the Meteorological Observer at the University of Texas states that the mean for the day is about 66%, the method of arriving at the mean not being given. This value agrees closely with the 68% used by the writer.

Another uncertainty is the height above the ground at which the wind velocity is measured, and the exact exposure of the evaporation pan as compared with the point at which the wind velocity is measured. As an additional example of the effect of wind on the rate of evaporation two records at Singapore may be cited. The Water Engineer's Department maintains two similar evaporation pans not very far apart, the mean annual evaporation for the same period being 52.95 in. for one and 42.33 in. for the other. The explanation for this difference is that one station is less exposed to the wind than the other.

The monthly records of evaporation which are deposited in Engineering Societies Library, in New York, N Y., give all available information regarding the various factors, but it was obviously impossible to present the details in the yearly summaries published in the writer's paper. The monthly records also give the source of each record.

Another reason for apparent discrepancies in some instances probably may be charged to human fallibility, and lies in errors in the observed records themselves; but, undoubtedly, there are errors in the reduction factors used, as they probably do not apply strictly in every instance.

Even with the known limitations it was felt that the bringing together on as comparable a basis as possible (and that basis the one that the engineer needs in his use of evaporation records), all records known to the writer, was a worthwhile contribution to the subject of evaporation from the practical engineering standpoint.

Mr. Grunsky's criticism of the floating pan as a method of measuring evaporation is corroborated by the discrepant comparisons cited by the writer in his original paper. Two other comparisons have come to the writer's attention. In Turkestan, E. M. Oldekop, a Russian engineer, compared records of evaporation between floating pans and land pans, and came to the conclusion that a factor of 0.73 was required to reduce the records of the former to reservoir surface evaporation.[48] Near Paramaribo, Suriname (Dutch Guiana), Dutch engineers compared the evaporation between a circular tank of concrete, 5.25 ft in diameter and 2.5 ft deep, sunk in the ground and three concrete tanks which have a rectangular surface, 3.3 by 2.6 ft, suspended in the water at a distance of 2 miles from the ground tank, and found that the floating tanks gave results which were 89% of those of the ground tank. Assuming that the ground tank requires a coefficient of 0.95 to reduce it to reservoir surface evaporation the comparison indicates that the coefficient for the floating tanks is 1.06.[49] Table 17, Item 9(*a*) is a circular concrete pan, 6.3 ft in diameter and 2.95 ft deep, sunk in the ground, with the top 4 in. above the surface and a coefficient of 0.95. As far as the writer has been able to learn, practically no attempts have been made in foreign countries to compare evaporation records from pans with those of larger bodies of water, and Sleight's classic experiments at Denver, Colo., are still the nearest approach to it that has been attempted.

From what has been stated, it is not surprising that Mr. Randell found discrepancies in the records presented by the writer, and it is obviously impossible to reduce them to one standard of meteorological factors, as much as such a procedure might be desirable. Other commentators have called attention to variations in these factors within a comparatively short distance of the evaporation pans, thus further emphasizing the fact that they are not directly comparable in many instances. The various comments are most valuable in bringing out the fact that evaporation records, even when carefully taken, are subject to considerable error and cannot be accepted as exact.

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[48] "The Evaporation from the Surface of the River Syr-Darya According to Observations at the Station Saporozskaya," *Journal of Irrigation No. 1,* 1924, Tashkent.

[49] *De Waterstaats Ingenieur.* May, 1924, No. 5, Batavia, Java.

It may be true, as suggested by Mr. Santos, that temperature, relative humidity, and wind velocity are not the only factors influencing evaporation, but in the vast majority of the existing records these are the only factors known and, in many instances, they are known only imperfectly.

Mr. Houk calls attention to the fact, recognized by the writer, that in arid regions, where large reservoirs are most likely to be constructed, relative humidity of the atmosphere adjacent to the water surface will be considerably higher than before such construction, with a corresponding decrease in evaporation. In humid regions, however, the effect on relative humidity will be much less marked. The writer cannot agree with Mr. Houk in thinking that the coefficient of 0.69 for the Yuma Citrus Station is much too high to show the actual evaporation under existing conditions. As Mr. Houk stated in a previous discussion,[50] the evaporation station known as Yuma Citrus Station is situated on a desert mesa where the (unmeasured) relative humidity must be very much less than that at the Yuma Date Orchard Station situated in an irrigated alfalfa field. The measured wind velocity is nearly twice as great at the Citrus Station as to the Date Orchard Station. Both these conditions are conducive to greater evaporation at the former station. However, if a reservoir were constructed at the former site the relative humidity would be increased and the evaporation decreased, although it appears to the writer that the greater wind velocity would still cause a higher rate of evaporation than in the more sheltered Date Orchard Station.

Evaporation Records from Foreign Countries.—Since transmitting the original paper, correspondence has been carried on with Government officials in every country where evaporation records might be available, and, as a result, nearly 200 foreign records have been obtained, together with records of temperature, wind velocity, and relative humidity, where these were available. These latter records may not be directly comparable in each instance. The records may be divided into three classes: (1) Records taken by means of equipment for which reduction factors to reservoir evaporation are fairly well known: (2) records from the Wild balance, an instrument widely used in foreign countries and for which some comparative results with standard equipment are available; and (3) records from Piche tubes.

The records in Class (1) have been reduced to reservoir evaporation, and the summaries are presented in Table 17. Those for the Wild balance (Class (2)) are presented in Table 18 as observed, but the Piche records (Class (3)), being taken under such widely varying conditions and appearing so discrepant among themselves, are not presented. The monthly records themselves have been deposited in the Engineering Societies Library.

The records of reservoir evaporation in Table 17 are taken by equipment described briefly in Table 19. The three concrete boxes at Tansa Lake in Bombay, India (Table 19, Item (24)), are set at three different levels and remain submerged after the end of the monsoon. They are exposed to the atmosphere in turn as the lake level recedes. At Bhatgar (Table 19, Item (26)), the iron tank is set on a stand in a circular concrete tank, 15 ft in

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[50] *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 282.

TABLE 17.—Summary of Evaporation Records Reduced to Reservoir Surface Evaporation

Item	Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind velocity, in miles per hour	Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches		Coefficient for pan
							April-September*	Annual	
	(a) Mexico								
1	Tacubaya (City of Mexico)	7 575	10	58	5.6	62	21.71	39.35	0.69
2	Presa Calles, Aguascalientes	6 644	10	62	..	..	29.31	49.81	0.69
3	Guadalajara, Jalisco	5 194	10	66	3.8	57	37.79	64.85	0.69
4	San Buenaventura, Chihuahua	5 164	10	62	..	..	37.26	58.31	0.69
5	Jalapa, Vera Cruz	4 590	10	64	2.5	78	20.73	37.93	0.69
6	Concordia, Coahuila	3 625	10	69	..	..	47.18	72.76	0.69
7	Don Martin, Coahuila	787	10	....	..	..	52.20	74.42	0.69
8	Santa Rosalia, Tamaulipas	.....	10	72	..	..	46.02	67.36	0.69
	(b) South America								
9	Georgetown, British Guiana	.....	1916–31	80	3.2	80	27.96	55.94	0.95
9 (a)	Paramaribo, Dutch Guiana	.....	1	80	2.0	83	24.21	49.03	0.95
	(c) West Indies								
10	Kingston, Jamaica	100	1924–30	79	7.6	79	33.07	55.54	0.75
	(d) Europe								
11	Lake Hjalmaren, Sweden[2]	315	1889–1929	42	10.2	80	12.58	14.98	0.83
12	Talla Water, Scotland	904	7	....	..	..	11.47	13.33	0.95
13	Glencorse, Scotland	639	1862–1904	47	..	81	11.44	13.97	0.95
14	Reevsby, Lincoln, England	124	6	....	..	..	10.16	13.68	0.95
15	Southport, Lancashire, England	37	7	49	..	82	10.70	12.23	0.80
16	Ormsby, St. Michael, Norfolk, England	21	7	48	..	..	13.46	16.56	0.95
17	Lower Laithe, Yorkshire, England	772	7	....	..	..	11.87	13.08	0.95
18	Ardsley, 1, Yorkshire, England	395	7	....	..	..	13.66	16.09	0.95
19	Ardsley, 2, Yorkshire, England	326	7	....	..	..	12.29	14.14	0.95
20	Harrowgate, Yorkshire, England	589	7	48	..	..	14.28	18.15	0.95
21	Petersfield, Hampshire, England	747	7	51	..	..	10.99	13.24	0.83
22	Otterbourne, Hampshire, England	112	7	50	10.7	..	15.36	18.22	0.95
23	Derwent Dam, Derbyshire, England	.....	1906–12	....	..	..	10.44	13.08	0.95
24	Camden Square, London, England	111	1885–1931	50	0.5	79	12.84	14.73	0.95
25	Lee Bridge, London, England	.....	1860–73	50	..	..	13.20	17.18	0.83
26	Kennick, South Devon, England	835	7	....	..	..	15.17	17.72	0.95
27	Plymouth, England	755	1907–31	49	..	..	12.42	16.10	0.95
28	Emdrup, Denmark[3]	.....	1849–59	45	..	..	19.34	23.17	0.83
29	De bilt, Holland[3]	10	1910–27	48	9.0	81	21.32	26.83	0.83
30	Agerisee, Switzerland	2 352	1912	51	..	..	18.71	29.15	1.00
31	Zugersee, Switzerland	1 368	1912	53	..	78	19.46	30.49	1.00
32	Grimnetzsee, Germany	.....	1909–13	48	5.5	78	24.10	29.16	0.83
33	Marathon Reservoir, Greece	830	1926–32	63	4.4	63	47.57	59.51	0.75
34	Marathon Reservoir, Greece	728	1926–32	63	4.4	63	48.11	62.72	0.83
	(e) Africa								
35	Cleveland Dam, South Rhodesia	.....	1913–31	65	..	60	39.41	71.04	0.83
36	Kopje Alleen, Transvaal	3 488	1926–27	66	..	61	44.83	72.75	0.95
37	Hartebeest, Poort, Transvaal	4 000	1926–30	66	..	63	42.83	67.51	0.95
38	Johannesburg, Orange Free State	5 925	....	60	12.5	66	35.12	62.36	0.95
39	Grootfontein, Cape Province	4 130	1926–27	62	..	59	50.77	77.93	0.95
40	Grassridge Dam, Cape Province	3 500	1926–30	62	..	65	51.97	76.92	0.95
41	Lake Arthur Reservoir, Cape Province	4 000	1926–30	....	..	..	47.93	71.73	0.95
42	Graaff Reinet, Cape Province	2 463	1926–30	65	..	59	53.29	79.94	0.95
43	Kamanassie Dam, Cape Province	1 100	1926–30	66	..	71	50.02	71.75	0.95
44	Lake Mentz, Cape Province	800	1926–30	67	..	69	48.42	68.26	0.95
45	Steenbras Reservoir, Cape Province	1 130	1922–31	59	..	66	39.83	53.80	0.95
46	Molteno Reservoir, Cape Province	310	1922–31	....	..	..	40.29	54.10	0.85
47	Windhoek, South West Africa	5 676	1928–31	66	7.6	30	57.07	94.95	0.94

* For stations in Southern Hemisphere, period is October to March.

TABLE 17.—(*Continued*)

Item	Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind velocity, in miles per hour	Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches		Coefficient for pan
							April-September*	Annual	
	(*f*) ASIA AND ISLANDS								
48	Saporozskaya, Turkestan	995	1912–17	58	10.0	55	.....	54.35	0.83
49	Kerki, Turkestan	879	1911–17	62	..	61	38.69	53.14	0.83
50	Tsining, Shantung, China	.....	1915–18	....	..	..	40.86	56.94	0.83
51	Shasi, Hupeh, China	.....	1929	64	..	..	23.25	30.96	0.83
52	Tuanfeng, Hupeh, China	.....	1929–30	64	..	..	23.96	33.43	0.83
53	Yukwanshan, Hupeh, China	82	1929–30	64	..	..	13.04	20.17	0.83
54	Anking, Anhwei, China	40	1929	67	..	..	24.73	35.69	0.83
55	Nanchang, Kiangsi, China	.....	1929	65	..	..	26.46	39.32	0.83
56	Kiukiang, Kiangsi, China	65	1929–30	62	..	..	25.51	35.79	0.83
57	Siangyin, Hunan, China	.....	1929	66	..	..	18.59	26.77	0.83
58	Singapore, Straits Settlement	.....	1927–31	81	..	81	26.34	52.95	0.75
59	Wadoek, Soembersono, Java	197	1914–23	79	..	80	27.34	53.39	0.83
60	Seoul, Korea	.....	1907–26	52	4.2	70	23.36	32.51	0.75
61	Sapporo, Hokkaido, Japan[1]	.....	1907–26	44	7.6	79	21.84	30.73	0.75
62	Osaka, Japan[1]	.....	1907–26	59	7.5	74	26.66	39.32	0.75
63	Tokyo, Japan[1]	.....	1907–26	57	6.0	74	18.51	29.34	0.75
64	Tainan, Formosa	.....	1907–26	73	7.1	80	28.92	50.36	0.75
65	Tansa Lake, Bombay, India	300	1906–21	81	..	78	24.08	50.71	0.85
66	Khadakwasla, India	.....	1924–32	....	..	..	27.10	56.41	0.83
67	Bhatgar, India	319	1917–32	....	..	..	29.35	65.49	0.88
	Mettur Reservoir, Madras, India	.....	1	84	4.9	82	38.48	72.18	0.69
	(*g*) AUSTRALIA								
68	Rockhampton, Queensland	37	14	73	..	67	28.05	46.96	0.90
69	Brisbane, Queensland	137	22	69	6.1	68	32.47	50.37	0.90
70	Cataract River, New South Wales	520	....	....	..	..	20.64	29.07	0.82
71	Sydney, New South Wales	138	51	63	8.5	70	22.20	32.01	0.82
72	Bathurst, New South Wales	2 323	16	57	3.2	71	29.11	39.62	0.82
73	Lake George, New South Wales	2 267	27	....	..	..	17.96	25.93	0.90
74	Hay, New South Wales	310	14	63	..	60	31.41	41.02	0.90
75	Leeton, New South Wales	466	16	62	..	60	36.90	47.73	0.90
76	Griffith, New South Wales	420	7	62	..	62	40.69	53.62	0.90
77	Yenda, New South Wales	432	4	62	..	60	40.46	52.88	0.90
78	Walgett, New South Wales	436	19	68	2.4	58	30.33	42.92	0.82
79	Hume, New South Wales	566	7	59	2.5	72	34.46	44.57	0.90
80	Wilcannia, New South Wales	267	25	66	..	52	37.51	52.70	0.82
81	Burrinjuck, New South Wales	1 239	18	58	2.2	70	25.04	32.30	0.90
82	Prospect, New South Wales	203	25	....	..	..	24.96	36.32	0.82
83	Lake Victoria, New South Wales	93	4	62	5.4	62	42.90	58.66	0.90
84	Umberumberka, New South Wales	900	20	66	10.3	58	57.34	80.21	0.90
85	Canberra, New South Wales	1 837	10	56	3.0	69	31.58	41.66	0.90
86	Wyuna, Victoria	323	4	54	..	67	29.78	35.77	0.90
87	Werribee, Victoria	79	19	58	..	72	30.21	41.31	0.90
88	Rutherglen, Victoria	533	19	60	..	66	34.66	43.91	0.90
89	Merbein, Victoria	185	10	62	..	66	46.21	63.15	0.90
90	Melbourne, Victoria	115	58	57	8.8	68	26.36	35.25	0.90
91	Adelaide, South Australia	140	61	63	9.7	55	37.88	49.45	0.90
92	Glen Osmond, South Australia	402	7	61	..	66	39.58	55.40	0.90
93	Alice Springs, South Australia	1 901	37	70	..	37	57.85	86.08	0.90
94	Marble Bar, West Australia	595	15	82	..	40	55.92	89.78	0.90
95	Coolgardie, West Australia	1 389	20	65	..	54	55.57	76.14	0.90
96	Merriden, West Australia	1 046	17	63	..	..	56.08	74.20	0.90
97	Narrogin, West Australia	1 114	15	60	..	..	37.87	50.94	0.90
98	Perth, West Australia	197	28	65	0.4	64	39.36	52.68	0.80
99	Hobart, Tasmania	177	20	54	7.3	68	20.83	28.63	0.90
100	Alexandra, New Zealand	520	1930–32	50	2.9	69	22.67	27.13	0.90

* For stations in Southern Hemisphere, period is October to March.

diameter and 7 ft deep. The concrete tank is set 2 ft in the ground. The evaporation records for this station are suspended during the monsoon, June to September, and for that period the evaporation has been estimated. At the Mettur Reservoir (Table 19, Item (27)), records are continuous throughout the year.

In Table 19, Item (29), a type of iron pan is described that rests on the bottom of a second iron pan. The latter is sunk in the ground with a 5-in. space between the sides of the two, which is filled with water. A similar arrangement is reported for Perth, West Australia, where the slate box (Table 19, Item (30)) is placed inside a concrete tank with a clearance of 5 in. on the sides and bottom.

The Wild balance consists essentially of a circular pan, 7 in. in diameter, resting on a balance. It is believed that the change in elevation of the pan itself, due to the lessening of the weight through evaporation, is indicated by a pointer on a curved scale.

The following comparisons between records from the Wild balance and other forms of evaporimeters have been noted: M. Audebeau-Bey, a member of the Institute of Egypt, has informed the writer that experiments carried on by him in Egypt during a period of two years in the center of the Nile Delta indicated that records from the Wild balance are about 67% of those from a Piche tube, both instruments being in a shelter with shuttered sides. The later work of the Egyptian Government engineers indicated that the coefficient to reduce Piche records in the same type of shelter to reservoir evaporation was 0.50. This coefficient indicates that the Wild records require a coefficient of 0.75 to reduce them to reservoir evaporation.

TABLE 18.—OBSERVED EVAPORATION BY MEANS OF WILD BALANCE IN SHELTER

Item	Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind velocity, in miles per hour	Relative humidity	OBSERVED EVAPORATION, IN INCHES	
							April–September	Annual
			(a) SOUTH AMERICA					
1	Cordoba, Argentine	......	1910–27	63	5.6	64	*30.80	46.39
			(b) EUROPE					
2	Bergen, Norway	138	1906–15	45	8.3	78	8.36	14.46
3	Stockholm, Sweden	143	1926–29	42	4.3	77	16.83	21.92
4	Lake Pyhäjärvi, Finland	260	1912	41	9.6	80	18.68	21.64
5	Bukarest, Roumania	......	1891–1905	51	......	67	20.48	26.73
6	Cadiz, Spain	33	1920–26	64	10.3	73	36.32	59.39
			(c) ASIA					
7	Saporozskaya, Turkestan	995	1915–17	59	8.9	52	48.34	66.99
8	Kawah Tjiwidei, Java	6 332	1917–18	57	......	91	10.44	20.37
9	Tosari, Java	5 692	1912–18	61	4.8	82	14.67	28.43
10	Tjibodas, Java	4 593	1912–18	64	......	87	12.01	24.19
11	Patjet, Java	3 642	1916–18	66	......	87	12.25	22.90
12	Bandoeng, Java	2 395	1912–18	71	......	81	20.80	40.42
13	Buitenzorg, Java	787	1913–18	77	......	83	20.67	36.25
14	Djember, Java	272	1913–18	77	......	82	24.37	45.04
15	Pekalongan, Java	30	1912–18	79	......	84	15.01	27.24
16	Batavia, Java	26	1912–22	79	2.5	83	11.30	21.72
17	Soerabaja, Java	16	1918–23	79	......	82	25.00	45.49
18	Pasoeroean, Java	16	1914–18	79	3.0	77	29.45	51.94

* Evaporation for period, October to March.

TABLE 19.—Description of Equipment in Table 17

Item (1)	Location (2)	Description (3)	Position (4)	Surface dimensions, in feet (5)	Depth, in inches (6)	Coefficient (7)
1	Mexico	U. S. Weather Bureau Class *A* equipment	....	....	....	0.69
2	Jamaica	Cylindrical copper pan (circular)	Above ground	0.67	4	0.75
3	Japanese Empire	Cylindrical copper pan (circular)	Above ground	0.67	4	0.75
4	Singapore	Cylindrical copper pan (circular)	Above ground	0.67	4	0.75
5	British Guiana	Concrete tank (water maintained 2 ft. deep)	Buried	6 by 6	....	0.95
6	Sweden	Floating pan (circular)	Floating	1.33	....	0.83
7	Great Britain	Galvanized iron pan	Buried	6 by 6	24	0.95
8	Great Britain	Galvanized iron pan	Buried	3 by 3	18	0.80
9	Great Britain	Galvanized iron pan	Buried	4 by 4	21	0.83
10	Great Britain	Slate box*	Floating	3 by 3	12	0.83
11	Denmark	Iron pan, painted white	On a grating in lake	1 by 1	8	0.83
12	Holland	Circular pan	Floating	0.75	....	0.83
13	Switzerland	Direct determination of evaporation by measured inflow and outflow, precipitation, and changes in lake flow. Gains or losses in lake bed believed to be negligible	....	....	....	....
14	Germany	Circular galvanized iron pan	Floating	1.67	20	0.83
15	Greece	Circular galvanized iron pan	Suspended from framework in reservoir	2.0	24	0.83
16	Greece†	Circular galvanized iron pan	Above ground	2.0	24	0.75
17	Union of South Africa	Galvanized iron pan	Buried	6 by 6	24	0.95
18	Union of South Africa	Cleveland Dam pan	Floating	3 by 3	18	0.83
19	Union of South Africa	Molteno Reservoir pan	In concrete embankment	4.5 by 2.5	18	0.85
20	Union of South Africa	Windhoek pan	Buried	6 by 6	18	0.94
21	Turkestan	Pan with area of 1.17 sq ft	Floating	....	....	0.83
22	China	Circular tin pan	Floating on water surface	2.0	12	0.83
23	Java	Concrete box	Suspended between poles in lake	3.3 by 3.3	30	0.83
24	Tansa Lake, Bombay, India	Three concrete boxes	Different levels	4 by 4	18	0.85
25	Khadakwasla, India	Iron tank	Floating	6 by 6	36	0.83
26	Bhatgar, India	Iron tank	On stand in circular concrete tank	6 by 6	36	0.88
27	Mettur Reservoir, Madras, India	U. S. Weather Bureau Class *A* equipment	....	....	....	0.69
28	Australia	Cylindrical iron pan	Buried	3	36	0.82
29	Australia	Cylindrical iron pan resting in another pan	....	3	36	0.90
30	Perth, West Australia	Slate box within a concrete tank	Buried	3 by 3	36	0.80

* At Lee Bridge, London, England. † Information supplied by R. H. Keays, M. Am. Soc. C. E.

The same writer stated that experiments carried on at the Aswan Reservoir justify the conclusion that the records of the Wild balance in a shuttered shelter are much nearer the evaporation from a reservoir surface than those of a Piche tube either in a shuttered shelter or in one with open sides. He did not give any details of those experiments.

At Helwan Observatory, near Cairo, experiments carried on by the Government engineers show the following relations between Piche and Wild records

(Piche in single shuttered shelter; Wild in double shuttered shelter): Winter ratio, 1.44; and summer ratio, 1.40.[51] By using the Piche reduction factor of 0.50, the coefficients to the Wild records are 0.72 during the winter months, and 0.70 during the summer months.

At Saporozskaya, Turkestan, direct comparisons between a Wild balance and floating pan have been made for the years 1914-15 and 1916-17.[52] By using 0.83 to reduce the floating-pan records to reservoir surface, the coefficients in Table 20 to reduce the Wild records have been obtained. The Wild balance was apparently placed in a shelter of the shuttered type.

TABLE 20.—COEFFICIENT TO REDUCE RECORDS OBTAINED BY WILD BALANCE

Year	Floating pan × 0.83, in millimeters	Wild balance, in millimeters	Coefficient
1914—15	1 028	1 354	0.76
1916—17	1 192	1 616	0.74

SUB-COMMITTEE ON EVAPORATION[53] (by letter).—The report of the Sub-Committee did not contain a preamble setting forth the argument leading up to the choice of pans. It was known that a great mass of data on pan evaporation existed. Likewise, it was known that the readings on these pans did not represent the evaporation that could be expected from a large water surface such as that of a reservoir. The most desirable pan to recommend appeared to be one that would co-ordinate with a pan of extensive use in the past and that was not too objectionable for use in the future. From this standpoint the Class *A* Weather Bureau pan was adopted after some discussion among the eleven members of the Special Committee on Irrigation Hydraulics.

The shortcomings of this pan were fully recognized—about in the same terms as set forth by Mr. Grunsky; and still it seemed the best. Although the coefficient is quite far removed from unity, the performance of this pan appeared to be the most consistent, with less chance of some unknown factor materially affecting results than would be the case with other types of pan. Of course, the floating pan with a coefficient of unity would be the best if the apparent readings could be assured as the true readings.

However, it was believed that the observer could never be certain that water had not been gained or lost over the rim during rough water on the surface containing the pan. For most floating pans it was believed that any guard sufficient to prevent wash of water over the rim would also be sufficient to inject a measure of doubt as to the relationship between evaporation in the pan and on the outside surface. It is possible that an outrider of brush, attached to the raft carrying the pan, can be made to ride so low in the water that it will reduce wave action and rocking of the raft and still not affect the wind conditions over the surface of the pan.

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[51] "The Nile Basin," Vol. 1, p. 58, Cairo, Egypt, Ministry of Public Works, 1931.

[52] *Journal of Irrigation, No. 1,* 1924, Tashkent.

[53] Fred C. Scobey, *Chairman,* Ivan E. Houk, R. L. Parshall, and Carl Rohwer, *Co-Operating Member,* Sub-Committee on Evaporation of the Special Committee on Irrigation Hydraulics.

The Sub-Committee adopts Mr. Grunsky's specification that the surface of the floating pan shall be maintained below that of the water outside the pan. It also agrees with him that a better pan than the Class *A* pan is desirable. However, it does not believe that the sunken land pan suggested by him should be recommended at this time. The Class *A* pan should be used in all future evaporation station installations until such a time as a satisfactory improved type of pan is developed and definitely proved to be better than the Class *A* pan for the purpose of securing long-time accurate records of evaporation.

If such a pan should be developed sometime in the future, it will be necessary to maintain records concurrently on both types of installation until the conversion factor for the two can be determined definitely. Such procedure would be imperative in order that the long-time records of Class *A* evaporation pan could be properly co-ordinated with records secured at the new type.

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Paper No. 1872

DEVELOPMENTS IN REINFORCED BRICK MASONRY

BY JAMES H. HANSEN[1], JUN. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. W. K. HATT, C. T. SCHWARZE, J. W. MCBURNEY, E. G. WALKER, AND JAMES H. HANSEN.

SYNOPSIS

This paper contains a brief history of the development of reinforced brick masonry to the time that engineers in the United States became interested in the subject. Its development in this country is then given in detail by analysis of tests.

Reinforced brickwork is a system of construction wherein reinforcing bars are placed in the joints of brick masonry (as shown in Table 1, Column (2)) so that they will take tensile stresses. Otherwise, brick masonry is built in the customary manner.

It is not the purpose of this paper to offer fully developed theories and conclusions, but to set down such information as is available to stimulate interest and discussion.

HISTORICAL NOTES

To that brilliant engineer, Marc Isambard Brunel, is generally conceded the credit for the invention of reinforced brick masonry. He was knighted by Queen Victoria in recognition of his greatest engineering feat, the Thames Tunnel.

The first step in connection with the building of the tunnel was the construction of a shaft, which Mr. Richard Beamish (who was Resident Engineer on the work), describes[2] as being 50 ft in diameter and 42 ft in height. It had forty-eight wrought-iron bolts 1 in. in diameter built into the brickwork,

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NOTE.—Published in March, 1933, *Proceedings*.

[1] Executive Secy., Brick Mfrs. Assoc. of New York, New York, N. Y.

[2] "Memoir of the Life of Sir Marc Isambard Brunel", p. 211; see, also *Civil Engineering*, September, 1931, Vol. 1, No. 12, p. 443.

which were attached to wooden curbs at the bottom and top with nuts and screws, "thus binding the work into a solid mass". Iron hoops and timber bonds, 9 in. deep and $\frac{1}{2}$ in. thick, were also let into the brickwork at intervals as work on the shaft progressed. When it had attained the height of 42 ft, the shaft was allowed to sink into the earth by excavating the gravel from the interior. Its strength and cohesion were severely tested when it suddenly descended 7 in. on one side where it had entered some soft ground, while on the other side, where the ground remained hard, it sank only $3\frac{1}{2}$ in. "The surge was alarming", says the annotator, "but so admirably was the structure bound together that no injury was sustained". After being sunk, the shaft was completed to the required depth by underpinning, but the shaft for the opposite shore was constructed in the same manner as the first, for its entire height of 70 ft. The second shaft differed from the first one in being slightly conical to facilitate its sinking, and it was more liberally supplied with reinforcing material.

According to Mr. Beamish the fortunate combination of iron ties with bricks and cement in the construction of the shaft of the Thames Tunnel suggested to Mr. Brunel the idea of establishing a general principle of construction "which should unite in an eminent degree utility with economy". Accordingly, after a series of experiments, he erected a semi-arch in the tunnel yard, without the use of centering, to a half-span length of 60 ft, and with a rise of only 10 ft 6 in. He counterbalanced it with weights totaling 62 700 lb. In the first part of this experiment, ties of fir, and, subsequently, hoop iron, were used. The arch stood for two years when it collapsed because some one dug too close to its shallow foundations. About the same time, Sir Isambard built what was later known as the "Nine Elms Beam" which was 57 in. deep, with an inverted **T**-section, 24 in. wide for the lower 18 in. and 19 in. wide for the upper part. This beam had a clear span of 21 ft and contained seventeen $1\frac{1}{4}$-in. by $\frac{1}{16}$-in. iron bands, on which was placed a load of 24 000 lb. It stood for two years when it was tested to destruction, in 1838, at a load of 68 326 lb, including the dead load. Failure was caused by the weakening of the iron straps, most of which were broken.

By this time other engineers were experimenting with this type of construction. In 1837, Col. C. W. Pasley, of the Royal Engineers, conducted a series of tests on brick beams, and in 1851, at the London Building Exhibition, a beam somewhat similar to the "Nine Elms Beam" was tested as one of the first demonstrations of the strength of Portland cement. This beam carried about five times as great a load as the "Nine Elms Beam", the latter having been built with Roman cement. It is interesting to note that these early tests were conducted to determine the quality of the various cements and mortars; the experimenters evidently did not question the adaptability of brick for this type of construction. Mention of its use is made in English publications in connection with several large buildings constructed at that time, and Isambard Kingdom Brunel, son of Sir Isambard Brunel, built one of his most beautiful brick arch bridges with hoop iron set in the brickwork in the manner which his father's experiments had proved to be successful.

From this stage it was only a step to the development of reinforced concrete, and this step taken, reinforced brickwork, which had played such a large part in introducing reinforced concrete, was allowed to languish and was practically forgotten. Mention was made of it now and then in English technical publications[3] during the last hundred years, but it was not until 1922 that it emerged from obscurity. In that year, A. Brebner, Under Secretary, Government of India, Public Works Department, published two volumes entitled "Notes on Reinforced Brickwork"[4], from which it is learned that this type of construction had had a rather wide use in India, where its development was due in part to the high cost of form lumber and concrete material. There was a lack of workmen familiar with concrete construction, whereas there was an abundance of brick and brick-layers. Several of the local governments in India have adopted it in the construction of public buildings and it has been utilized in the construction of many private buildings. In all, nearly 3 000 000 sq ft of reinforced brickwork was laid in the three years prior to 1922. Mr. Brebner's volumes include reports of tests conducted on numerous types of slabs, beams, cantilevers, etc., and, as a basis of comparison, similar tests on reinforced concrete structures, from the results of which he concluded that the theory of reinforced concrete design would apply to reinforced brick masonry.

In 1924, Capt. T. W. D. Miller, of the Royal Engineers, published a paper[5] written from notes made on the work done in the erection of an officers' hostel at Chaklala, India, in which he describes in detail the design and construction of the roof slabs of reinforced brick masonry. The slabs were originally designed for tile and reinforced concrete, but it was considered that the cost would be reduced by substituting reinforced brick masonry. The design is based on the standard reinforced concrete formulas with compressive stresses in the masonry limited to 600 lb per sq in. and tensile stresses in steel to 14 000 lb per sq in.

Dr. Engr. Shigeyuke Kanamori, Civil Engineer of the Department of Home Affairs of the Imperial Japanese Government, writing on the subject of reinforced brick masonry[6] states that he has built many structures, such as buildings, retaining walls, quay walls, bridge piers, abutments, etc., of reinforced brick masonry. His design practice is to use reinforced concrete formulas with the value of 25 for n, the ratio of elasticity moduli, and brick made with a slot in one side which always coincides with the slot below, providing space for the rods.

Characteristics of Brick and Brick Masonry

The books and articles cited are recommended for perusal, but are not taken into consideration in this paper, as its object is the discussion of the technical phases of reinforced brick masonry as developed in the United States. It has been considered best in this country to make independent experiments so as to avoid the difficulties incident to allowances for differences in climatic conditions and materials, the principal difference being that the brick in the United States are smaller and stronger than those made in India and Japan. The tests and experiments discussed in this paper were by no means ideally planned, but they include a variety of beams and slabs over a number of span lengths, and, for practical purposes, they establish the truth of certain necessary assumptions and values that can be used with safety subject to further development. The necessary determinations were: (*a*) The compressive, tensile, and shear strengths of brick, mortar, and brick masonry; (*b*) the modulus of elasticity of brick masonry; (*c*) its resistance to shear; and (*d*) the bond strength developed by bars in mortar joints.

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[3] *Engineering* (London), July, 1872.

[4] *Technical Paper No. 38*, Govt. of India, Public Works Dept.

[5] *Royal Engineers Journal*, Vol. 38, p. 108.

[6] *Brick and Clay Record*, July 15, 1930.

Compressive Strength.—The Building Code Committee of the United States Department of Commerce has distributed tables[7] of brick masonry strengths upon which its allowable stresses were based. The maximum compressive strength listed was 18 337 lb per sq in., the minimum, 1 659 lb per sq in., and the average of all is 6 940 lb per sq in. Only 27% had compressive strengths of 3 500 lb per sq in., or less, and these average 3 115 lb per sq in.[8] The National Bureau of Standards has tabulated other properties of brick[9], which show that: (1) The direct ratio of compressive strength, flat, of a half brick to compressive strength on edge is 1.08; and (2) the inverse ratio of compressive strength, flat, to the modulus of rupture is 0.24; to tensile strength, 0.09; and to shear strength, 0.39. These are averages of fifty tests of brick from four different parts of the United States. However, there is a considerable variation of these ratios for brick in different territories.

Handbooks and test reports give widely different values for the compressive strength of brick masonry, which, considering the number of variables, is only to be expected; but it can be controlled and predetermined within limits just as it is in other construction materials where these factors are known. The size of the specimen tested, the type of workmanship, the mortar, and the brick strength are of prime importance for any direct comparison. Investigators of brick masonry have generally confined themselves to tests on individual bricks, "wallettes", approximately 18 by 34 in. high, and wall panels, 6 by 9 ft high, with a few tests on small piers of various sizes, some as small as 8 by 12 in. Which of these specimens gives results most applicable to the strength of masonry in beam action has not been determined, but perhaps a brief summary of them all will suggest an answer.

In general, if all other things are equal, the relation between strength of the masonry and strength of brick is a straight-line ratio until the ultimate strength of mortar is reached. The Bureau of Standards reports[10] the average strength of "wallettes", 18 by 34 in. high, with 1:0.1:3 cement mortar, to be 2 110 lb per sq in. It reports[11] the strength of walls to be 86% of the strength of "wallettes", and it is generally conceded that walls 9 ft high will develop a compressive strength equal to from 25 to 30% of the brick strength. When small piers $\left(\frac{\text{height}}{\text{diameter}} = 2\right)$ are tested, the strength of brick masonry laid in cement mortar will no doubt be greater. After a study of hundreds of tests of the nature of the foregoing, the Building Code Committee of the Department of Commerce has recommended[12] that the allowable working stress in direct compression on the best brick masonry, when properly designed, should be increased to 600 lb per sq in. Allowable working stresses at present seldom exceed 350 lb per sq in.

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[7] "Compression Tests on Brick Masonry": A Compilation of Data Covering 708 Individual Tests of Brick Masonry in the Form of Piers and Walls", March, 1926.

[8] Discussion by Maj. L. B. Lent of the proposed recommendations of the Building Code Committee, U. S. Department of Commerce, May 25, 1928. (Not published.)

[9] "Compressive Strength of Clay Brick Walls", by A. H. Stang, D. E. Parsons, and J. W. McBurney, *Research Paper No. 108*, National Bureau of Standards.

[10] *Research Paper No. 108*, National Bureau of Standards, U. S. Dept. of Commerce, Table 23.

[11] *Loc. cit.*, Fig. 35.

Modulus of Elasticity.—The modulus of elasticity of brick masonry has been variously determined as follows:

Authority	Modulus of Elasticity, in Pounds per Square Inch
T. G. MacCarthy[13]	3 000 000
A. H. Beyer, M. Am. Soc. C. E., and W. J. Krefeld[14] Assoc. M. Am. Soc. C. E.	1 628 830
Mason Vaugh[15]	3 171 000
M. J. Feeney and R. W. Miller[16]	1 350 000
J. G. Bragg[17]	580 000 to 3 500 000

The values cover ranges of mortar mixes generally used.

Messrs. A. H. Stang, D. E. Parsons, and J. W. McBurney have stated[18] that there is no value for the modulus of elasticity that is constant over a large stress range, but they give calculated values for the secant modulus over a stress range of 0 to 250 lb per sq in. for cement mortar walls to be from 727 000 lb per sq in. to 2 652 000 lb per sq in. These variations are not alarming and can be controlled more or less as with concrete, which has a variation of 750 000 lb to 3 000 000 lb, resulting in values of n of from 10 to 40.[19]

Tensile Strength.—Little has been published concerning the tensile strength of brick masonry; it is limited in the case of cement mortar by the bond strength between brick and mortar and in the case of cement-lime mortar by the tensile strength of the mortar itself. Messrs. L. A. Palmer and J. V. Hall give[20] bond strengths of 31.7 lb for 1:1:6 cement-lime mortar and approximately the same value for 1:3 cement mortar. Another authority[21] gives 10 to 40 lb (including a factor of safety of from 8 to 10) as the tensile strength of brick masonry. Many authorities agree that the more porous bricks develop higher bond strengths, provided they are properly wetted before laying. Ordinary bricks that are very dry will absorb the water from the mortar too rapidly, and those completely saturated lack the suction necessary to cause the mortar to cling to them. This is also slightly noticeable with dense, hard bricks of low absorption.

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[12] "Modifications in Recommended Minimum Requirements for Masonry Wall Construction", March 16, 1931.

[13] Rept. on Brick Masonry, Southern Methodist Univ. (Not published).

[14] *Engineering and Scientific Papers No. 12,* Columbia Univ., April, 1923, p. 58.

[15] *Bulletin No. 37,* Univ. of Missouri, Vol. 29.

[16] In a thesis presented to Mass. Inst. Tech. in partial fulfillment of the requirements for the degree of Bachelor of Science in Architectural Engineering.

[17] *Technologic Paper No. 111,* National Bureau of Standards, U. S. Dept. of Commerce.

[18] *Research Paper No. 108,* National Bureau of Standards, Washington, D. C.

[19] Concrete Engineer's Handbook, by Hool and Johnson, Vol. I, p. 6, First Edition, 1920.

[20] *Research Paper No. 290,* National Bureau of Standards, Washington, D. C.

[21] Kidder's Architects and Builders Handbook, Seventeenth Edition, 1921, p. 179.

Comparison of Test Data

Referring to Table 1, complete information regarding the conditions of each test was not always available and certain assumptions have been made; but, in general, all data that would materially affect the results obtained were taken from the test reports. The calculations are based on the assumption that the value of 20 for n, the ratio of elasticity moduli, would give a fairly accurate distribution of stresses, and that the theories of beam action in composite construction, as defined in the report of the Joint Committee on Standard Specifications for Reinforced Concrete[22], would apply. Due to the methods used in some of the beam tests, the testing apparatus itself placed a load of from 240 lb to 500 lb on the beam. However, as the true weight of this item was not reported in all cases, it was not considered; only live loads are listed. The weight of the beam itself was also disregarded in calculating the bending moment, and stresses in the steel and brickwork. The inclusion of these items would raise the stresses, but not enough to affect the conclusions as to the stability of the beams. In calculating the shear, unit shear, and bond (Columns (6), (9), and (10), Table 1), however, the weight of the beam was calculated at 125 lb per cu ft and used because these stresses are critical and it is important to know the maximum value that can be developed with safety. In Table 1 the calculated stresses are placed opposite the loads to which they apply, which are: (1) Load at first crack or at a deflection of $\frac{1}{360}$; (2) load at which the last deflection was measured; and (3) the ultimate load.

Items Nos. 1 to 6, Table 1.—The summary of test results by Mr. Vaugh pertains to a group of beams with clear spans of 12.00 ft. The distance between the two axle loads was 4.00 ft in each case. Three of the beams were made up of a 1:0.0:2 cement-sand mortar (Items Nos. 1, 2, and 3) and were tested at 49, 44, and 42 days, respectively. The remaining three (Items Nos. 4, 5, and 6), were made up of a cement-lime-sand mortar in the ratio of 1:0.1:3 and were tested at 41, 40, and 46 days, respectively. In Items Nos. 4 and 5, $\frac{1}{4}$-in. round bars—and in Item No. 6, No. 9 wire—spaced 8.5 in., on centers, were used as stirrups. Two $\frac{3}{4}$-in. round, plain bars were used for reinforcement as shown in Column (2). All rods were hooked except those in Item No. 1.

Mr. Vaugh states[15] that only one beam (Item No. 3) failed by shear (at 69 lb per sq in.). The other two with no shear reinforcement failed in bond, or tension in the longitudinal steel, despite the fact that shear stresses were as much as 85 lb per sq in. (see Item No. 1). However, when stirrups and cement-lime mortar were used in the same type of beam (Items Nos. 4, 5, and 6), the failure was due to tension in all cases. In two cases (Items Nos.

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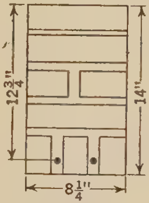
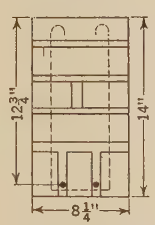
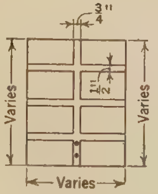
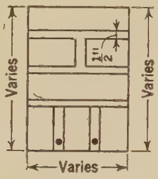
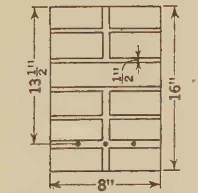
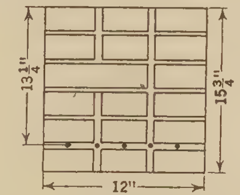
[22] *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

TABLE 1.—CHARACTERISTICS OF REINFORCED BRICK BEAMS AND RESULTS OF TESTS (ALL BEAMS FREELY SUPPORTED AT ENDS)

Item No.	Cross-sections	Live load, in thousands of pounds (N = total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, V in pounds	STRESSES, IN POUNDS PER SQUARE INCH			
						Tensile stress, f_s, in reinforcement	Compression stress, f_b, in beams	Shear, v	Bond strength, u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	MASON VAUGH								
1		7.0	93	....	......	......	......	....	.....
		14.0	309	336	7 672	34 900	1 495	85	150
2		4.0*	30	....	......	......	......	....	.....
		7.0	84	168	4 172	17 400	670	46	82
3		7.0*	84	....	......	......	......	....	.....
	Item Nos. 1 to 3.	11.0	182	264	6 172	27 400	1 050	69	112
4		7.0*	91	....	......	......	......	....	.....
		17.0†	364	407	9 172	42 300	1 620	102	180
5		7.0‡	194	....	......	......	......	....	.....
		11.2‡	533	268	6 247	27 800	1 060	70	122
6		7.0‡	89	....	......	......	......	....	.....
	Item Nos. 4 to 6.	17.0‡	387	407	9 172	42 300	1 620	102	180
	M. J. FEENEY AND R. W. MILLER								
7		9.5†	60	....	......	......	......	....	.....
		11.5	96	....	......	......	......	....	.....
		18.5	....	185	9 340	57 800	1 610	136	369
8		9.5†	128.5	90	4 675	33 100	820	84	214
		12.5	187.5	....	......	......	......	....	.....
		16.4	....	163.5	8 350	60 000	1 485	154	384
9		7.5†	33	75	3 925	26 000	643	72	170
		12.5	95	....	......	......	......	....	.....
		16.3	....	163	8 325	56 700	1 400	151	360
10	Item Nos. 7 to 11.	5.5†	126.5	88	3 030	25 400	712	43	109
	Item No. 7: 8½ by 10¼; $d = 9.25$.	7.5	227	....	......	......	......	....	.....
	Item No. 8: 8 by 9; $d = 8.25$.	10.1	....	162	5 330	46 500	1 320	76	192
11	Item No. 9: 7½ by 10; $d = 8.75$.	5.5†	109.5	88	3 030	31 700	985	53	136
	Item No. 10: 8 by 10¾; $d = 10.0$.	8.5	263.5	....	......	......	......	....	.....
	Item No. 11: 8 by 10; $d = 8.12$.	10.4	....	166	5 480	59 800	1 860	97	246

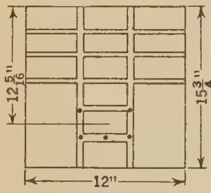
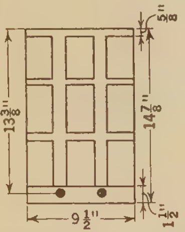
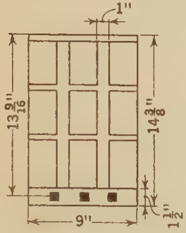
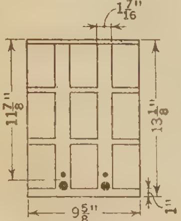
* No sign of failure. † Load at first indication of failure. ‡ Ultimate load.

TABLE 1.—(*Continued*)

Item No.	Cross-sections	Live load, in thousands of pounds (N=total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, V, in pounds	STRESSES, IN POUNDS PER SQUARE INCH			
						Tensile stress, f_s, in reinforcement	Compression stress, f_b, in beams	Shear, v	Bond strength, u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	M. J. FEENEY AND R. W. MILLER (*Continued*)								
12	 Item Nos. 12 to 16.	9.5†	28	95	4 950	25 700	680	66	167
		12.5	52	....	......	......	......	....	.....
		22.0‡	....	220	11 200	59 500	1 570	149	377
13		9.5†	....	95	4 950	24 800	646	64	162
		17.5‡	....	175	8 950	45 800	1 190	115	292
14		9.5†	42.5	95	4 950	24 800	646	64	162
		12.5‡	61	....	......	......	......	....	.....
		19.5‡	....	195	9 950	51 000	1 330	128	324
15		5.0†	54	80	2 740	21 400	566	36	69
		10.5	174.5	....	......	......	......	....	.....
		13.3‡	....	212	6 890	57 000	1 530	91	230
16	Item No. 12: 8 by 11½; d = 10.67.	1.5	3	....	......	......	......	....	.....
	Item No. 13: 8 by 11¾; d = 11.0.	1.5	5	....	......	......	......	....	.....
	Item No. 14: 8 by 11½; d = 11.0.	1.5	7	....	......	......	......	....	.....
	Item No. 15: 8 by 11½; d = 10.75.	3.8	45	....	......	......	......	....	.....
	Item No. 16: 8 by 11½; d = 10.75.	3.8	32	....	......	......	......	....	.....
		13.3	....	212	6 890	57 000	1 530	91	230
	JUDSON VODGES								
17	 Item Nos. 17 and 18.	5.4†	....	87	3 250	47 300	593	33	110
		7.6‡	....	121	4 314	65 900	823	43	146
18		4.9†	....	79	3 010	43 000	537	30	102
		7.5‡	....	120	4 272	65 000	813	43	145
19	 Item No. 19.	8.1†	....	129	4 837	43 300	599	33	101
		10.7‡	....	171.2	6 160	57 600	800	42	130

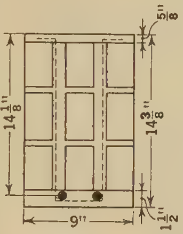
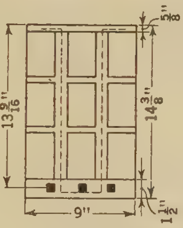
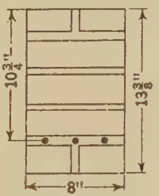
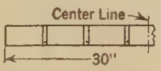
† Load at first indication of failure. ‡ Ultimate load.

TABLE 1.—(*Continued*)

Item No. (1)	Cross-sections (2)	Live load, in thousands of pounds (*N* = total uniform load; remainder are axle loads) (3)	Deflection, in thousandths of an inch (4)	Bending moment, in thousands of inch-pounds (5)	Total maximum shear, *V*, in pounds (6)	STRESSES, IN POUNDS PER SQUARE INCH			
						Tensile stress, f_s, in reinforcement (7)	Compression stress, f_b, in beams (8)	Shear, v (9)	Bond strength, u (10)
	JUDSON VODGES (*Continued*)								
20	 12 5/16" 15 3/4" 12" Item No. 20.	7.8† 13.0‡	....	125.2 208	4 712 7 300	44 800 74 300	666 1 108	35 54	105 162
	T. G. MACCARTHY								
21	 5/8" 13 3/8" 14 7/8" 9 1/2" 1 1/2" Item No. 21.	7.0 15.0 16.2	59 140	 259	.... 8 588	.... 23 800	 814	 78	 148
22	 1" 13 9/16" 14 3/8" 9" 1 1/2" Item No. 22.	7.0 16.2	54 160	.... 259.5	...... 8 580	...... 24 000	 905	 81	 98
23	 1 7/16" 11 7/8" 13 1/8" 9 5/8" 1" Item No. 23.	7.0 19.0 24.0‡	87 270 380	.... 304 384	...... 9 932 12 432	...... 30 300 37 950	...... 1 180 1 500	.... 112 140	..... 137 173

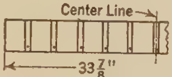
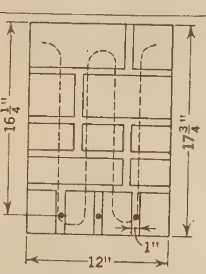
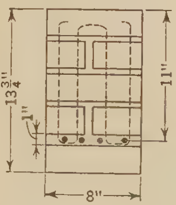
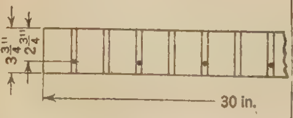
† Load at first indication of failure. ‡ Ultimate load.

TABLE 1.—*(Continued)*

Item No.	Cross-sections	Live load, in thousands of pounds (*N* = total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, *V*, in pounds	Stresses, in Pounds per Square Inch			
						Tensile stress, f_s, in reinforcement	Compression stress, f_b, in beams	Shear, *v*	Bond strength, *u*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	T. G. MacCarthy (*Continued*)								
24	 Item No. 24.	23.0† 26.5‡	250 339	368 425	11 988 13 738	33 700 39 000	1 169 1 350	109 125	208 236
25	 Item No. 25.	7.0 25.0 30.0‡	62 258 340	.... 400 480	...... 12 980 15 480	...... 36 900 44 300	...... 1 390 1 670	.... 123 147	..... 148 176
	Hugo Filippi								
26	 Item Nos. 26 to 31.	2.0† 2.5‡	125 344	....	....	....	....	....	....
27		3.0† 4.0‡	156 187	60.0 79.2	2 035 2 524	18 850 24 900	450 590	27 33	61 75
28		2.5† 3.3‡	94 125	50 66.6	1 735 2 201	15 700 20 950	374 500	23 29	51 65
29		2.5† 4.0‡	94 250	50.0 80.0	1 735 2 535	15 700 25 000	374 596	23 34	51 75
30		2.5† 7.5‡	94 219	50.0 150.0	1 735 4 285	15 700 47 300	374 1 120	23 56	51 126
31		3.5† 6.5‡	94 391	70.0 130.0	2 285 3 785	22 000 40 800	525 975	30 49	67 111
32	 Item Nos. 32 to 37.	2.0 *N*† 3.7 *N*‡	156 500	17.3 31.8	1 150 2 012	38 200 70 500	1 140 2 120	25 44	150 280
33		2.3 *N*† 4.2 *N*‡	156 563	19.3 33.9	1 285 2 260	42 750 75 000	1 280 2 230	28 49	178 314
34	Width = 30 in; depth of slab = 2¼ in.; effective depth, *d* = 1¾ in.	2.0 *N*† 3.8 *N*‡	125 594	17.3 30.8	1 150 2 048	38 200 68 200	1 140 2 032	25 45	150 285

† Load at first sign of failure. ‡ Ultimate load.

TABLE 1.—(*Continued*)

Item No. (1)	Cross-sections (2)	Live load, in thousands of pounds (*N* = total uniform load; remainder are axle loads) (3)	Deflection, in thousandths of an inch (4)	Bending moment, in thousands of inch-pounds (5)	Total maximum shear, *V*, in pounds (6)	STRESSES, IN POUNDS PER SQUARE INCH: Tensile stress, *fs*, in reinforcement (7)	Compression stress, *fb*, in beams (8)	Shear, *v* (9)	Bond strength, *u* (10)
HUGO FILIPPI (*Continued*)									
35		2.0 *N*† 3.4 *N*‡	125 625	17.3 27.7	1 150 1 845	38 200 61 400	1 140 1 830	25 40	150 256
36		2.2 *N*† 3.5 *N*‡	313 563	19.1 28.5	1 270 1 950	42 200 63 200	1 259 1 880	28 42	177 270
37		2.5 *N*† 3.8 *N*‡	219 531	21.0 30.4	1 400 2 025	46 500 67 400	1 390 2 000	30 44	192 282
38	 Item Nos. 38 to 43.	4.0 *N*† 6.3 *N*‡	344 938	58.7 85.5	2 440 3 567	33 800 45 800	955 1 207	26 38	123 182
39		4.0 *N*† 6.2 *N*‡	344 688	58.7 85.0	2 440 3 543	33 800 48 000	955 1 355	26 38	123 180
40		4.0 *N*† 7.5 *N*‡	313 1 000	58.7 107.0	2 440 4 190	33 800 57 800	955 1 630	26 45	123 212
41		3.5 *N*† 8.5 *N*‡	219 1 125	52.6 112.8	2 190 4 690	30 300 64 900	855 1 830	23 50	111 238
42		2.5 *N*† 8.4 *N*‡	78 875	40.6 111.0	1 690 4 626	23 300 59 400	659 1 560	18 48	86 350
43		3.0 *N*† 9.0 *N*‡	125 906	46.6 118.4	1 940 4 940	26 800 68 100	755 1 920	21 53	99 250
TESTS BY THE WRITER									
44		1.0	0	....	......	......	......	....	....
		3.1	32	....	......	......	......	....	....
		5.3	89	....	......	......	......	....	....
		7.4	150	....	......	......	......	....	....
		8.4	159	270.5	5 580	14 450	487	33	56
		9.5	220	....	......	......	......	....	....
		11.7	297	....	......	......	......	....	....
		13.8	380	....	......	......	......	....	....
		16.0	462	....	......	......	......	....	....
		18.0†	780	578.0	10 385	30 800	1 033	61	104
		18.0	....	....	......	......	......	....	....
		20.1	....	641.0	11 405	34 370	1 155	68	115
45		2.1	41	....	......	......	......	....	....
		3.4	87	....	......	......	......	....	....
		4.4	121	97.7	2 682	13 080	574	35	45
		6.2	200	....	......	......	......	....	....
		7.6	269	....	......	......	......	....	....
		9.1†	338	199.9	5 001	26 770	1 170	65	83
		10.4	415	....	......	......	......	....	....
		11.8	496	....	......	......	......	....	....
		13.9	1 000	....	......	......	......	....	....
		14.5	....	....	......	......	......	....	....
		15.9	....	....	......	......	......	....	....
		16.6	....	....	......	......	......	....	....
		21.3	....	469.0	11 056	49 500	2 138	144	183
46		0.3	0	23.7	......	......	......	....	....
		0.8	78	....	......	......	......	....	....
		0.9	104	....	841	18 000	602	12	60
		1.2	186	....	......	......	......	....	....
		1.8	314	....	......	......	......	....	....
		2.3	450	....	......	......	......	....	....
		2.6	560	....	......	......	......	....	....
		3.3	1 000	....	......	......	......	....	....
		3.4†	....	64.9	2 120	49 200	1 648	30	151
		3.9‡	....	72.4	2 360	55 000	1 837	33	168

† Load at first indication of failure. ‡ Ultimate load.

4 and 6) this occurred quite near the yield point of the steel. The bond stress (Column (10)) was as much as 180 lb per sq in. and the shear stress (Column (9)), as much as 102 lb per sq in.

Although the mortar was changed from a straight cement-sand to a cement-lime-sand mixture it seems that the higher stresses obtained were due mostly to the use of stirrups. This would indicate that failure in the case of Items Nos. 1 and 2 would also have been delayed if stirrups had been used. The six beams in this group failed gradually and were removed from the testing machine whole, indicating that a factor of safety can be established on the basis of gradual failure. In testing Item No. 6, the load was applied five times up to 14 000 lb, after which the permanent deflection was 0.036 in. and the elongation of the steel was 0.0006 in. in a gauge length of 8 in. Then the load was increased to 17 000 lb, causing a deflection of 0.387 in. The allowable deflection is 0.4 in.

It is unfortunate that the true widths of the joints were not recorded, because it is of prime importance to know how wide the mortar joint must be to develop sufficient bond. It seems improbable that they were wider than 1 in., which would provide for only $\frac{1}{8}$ in. of mortar between the $\frac{3}{4}$-in. rods and the brick. In other words, the mortar was in contact with the bars only at the top and bottom to all intents and purposes.

Important observations reported were: (*a*) The beams were built of shale bricks with an average compressive strength of 5 953 lb per sq in.; (*b*) the bricks were sprinkled in the pile; (*c*) no brick or mortar was crushed during the test; (*d*) no bottom bricks fell off; and (*e*) a very dry mortar was used.

Items Nos. 7 to 16, Table 1.—Of the tests listed the most comprehensive are those conducted by Messrs. Martin J. Feeney and Raymond W. Miller.[16] The tests consist of four groups of three beams each with two different cross-sections and two span lengths with one size of bar, one mortar mix, and one type of brick, tested to study the action of beams as the load increased. Tests 10, 15, and 16 had clear span lengths of 10 ft, and the distance between concentrated axle loads was 2 ft 8 in. The remainder of the beams had clear span lengths of 5 ft, and the distance between axle loads was 1 ft 8 in. A cement-lime mortar mix of 1:0.15:3 was used throughout, and each of the beams was tested 28 days after it was made. None of the beams reported was reinforced by stirrups. The longitudinal bars consisted of two $\frac{1}{2}$-in. round, deformed bars, all hooked at the ends.

The beams were built of sand struck common clay brick with a compressive strength on the flat side of 4 715 lb per sq in. The reinforcement bars were placed by means of hangers. The mortar in the beams was tested with the following results: Items Nos. 7 to 9, 1 126 lb per sq in.; Items Nos. 10 and 11, 1 138 lb per sq in.; Items Nos. 12 to 14, 1 005 lb per sq in.; and Items Nos. 15 and 16, 1 430 lb. per sq in.

The deflections entered under Item No. 16 require a further word of explanation. The first load of 1 500 lb caused a deflection of 0.003 in. After 24 hours, the load was released and applied again, causing a deflection of 0.005 in. This cycle was repeated twice in 24-hour intervals, with the results shown.

Finally, after six days, the load of 3 750 lb was released, and again applied, to cause a deflection of 0.032 in. The specimen was finally loaded to 13 300 lb, as indicated in Item No. 16, Table 1.

Although the experimenters found a considerable movement upward, of the neutral surface as the load increased, their calculations (for one beam only) showed fiber stresses and values of j and k to be quite close to those of a similarly reinforced concrete beam, and Messrs. Feeney and Miller concluded that reinforced concrete formulas can be used with reasonable assurance in the design of reinforced brick beams. The stresses computed for this group (Items Nos. 7 to 16) show values as high as 1 860 lb per sq in. in the brickwork and 60 000 lb per sq in. in the steel, and shear stresses as high as 84 lb per sq in. at the first crack, with a maximum shear of 154 lb per sq in.

A detailed description of failure is not given for all beams, but the three views of typical failures included in their report show cracks running along the mortar joints, and, in some cases, through the brick, at various angles and different sections of the beam. One of these beams (see Fig. 1) developed four vertical cracks between the third points, with a fifth crack running diagonally from the third point toward the end of the beam. Another (see Item No. 9, Table 1) had two vertical cracks quite close to, and on either side of, the center line. The third beam (Item No. 14, Table 1) showed a typical shear crack from the support to the third point. The experimenters declared that the primary cause for failure in each case was a weakening of the masonry in tension. However, vertical cracks near the center of the beam indicate either bond or shear failure, and as there was apparently only one

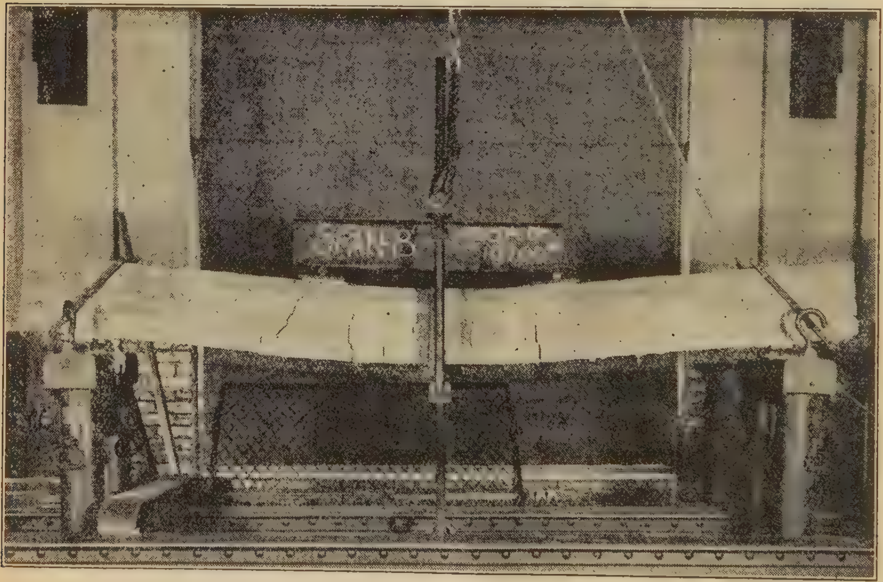


FIG. 1.—BEAM TESTED TO FAILURE BY MESSRS. FEENEY AND MASON (SEE ITEM NO. 11, TABLE 1).

beam showing a typical shear crack, it is logical to assume that in the cases where the writer's calculations show steel stresses to be 50 000 lb per sq in., or more, and bond stresses of 300 lb per sq in., or more, these stresses must have been the causes of failure.

Since the masonry is not expected to take tensile stresses the fact that it did not, can scarcely be given as the cause for failure. The greater bond and shear stresses are reported by Messrs. Feeney and Miller, but with too many variations in materials and construction to determine the cause. It is to be noted, however, that the bars were placed in vertical joints as in Items Nos. 1 to 6, and the mortar was almost the same as in Items Nos. 3 to 6, which appears to be significant.

Items Nos. 17 to 20, Table 1.—The tests recorded as Items Nos. 17 to 20 in Table 1 are largely self-explanatory. The beams were 10 ft 8 in. long, with a clear span of 8 ft. The distance between concentrated loads was 2 ft 8 in., and the mortar mix was 1:0.1:3 for Items Nos. 17 and 18, and 1:1:3 for Items Nos. 19 and 20. The beams had no stirrups and the longitudinal reinforcement consisted of $\frac{1}{4}$-in. round bars. The bars in Items Nos. 17 and 19 were straight and those in Items Nos. 18 and 20 were hooked on the end. Three bars were used in Items Nos. 17 and 18, and five in Items Nos. 19 and 20. Other tests of a similar nature have been conducted under the supervision of Mr. Judson Vogdes, District Engineer for the Common Brick Manufacturers Association of America.[23] These tests include one beam, 8 by 25 in., on a 20-ft span reinforced with five $\frac{3}{8}$-in. round bars that carried a load of 12 226 lb. The maximum deflection was $\frac{9}{16}$ in., or $\frac{1}{8}$ in. less than the allowable $\frac{1}{360}$ of the span length. Failure was considered to be due to tension in the steel because no sign of compression or shear failure was evident. Another series of tests were conducted on 4-in. slabs reinforced with a $\frac{3}{8}$-in. round bar in each joint, and resting on brick beams, 8 ft on center, cantilevered 4 ft over each beam. They were loaded uniformly to gradual failure, under the same cantilever and sustained center-span loads of 995 lb per sq ft. The first crack was a hair crack that appeared over the support at a cantilever load of 2 568 lb, or 350 lb per sq ft, and 648 lb per sq ft on the center span. At 880 lb per sq ft on the center span, a crack appeared across the bottom at the center. At the ultimate center load of 995 lb per sq ft, the deflection was $\frac{7}{32}$ in., or about $\frac{1}{360}$ of the span.

In other tests on the same type of structure, in which the mortar was 1:1:6 instead of 1:0.15:3, the ultimate load was about 825 lb per sq ft. Providing for a factor of safety of 3.5, the safe load for these slabs would be 284 lb. For such a load a reinforced concrete slab with an effective depth of 4.15 in. would be required on the basis of 800 lb per sq in. for compressive stresses in the masonry. The effective depth of the slab tested was $3\frac{1}{2}$ in. Available photographs of these tests, however, indicate an arch action in the loading material which, if true, would modify these results considerably.

Items Nos. 21 to 25, Table 1.—Professor T. G. MacCarthy has reported[13] tests of twenty brick slabs, six brick beams, one mortar slab, and one concrete slab. The mortar slab consisted of the same mix as the mortar used with the brick, which was 1:0.15:3. The concrete was a 1:2.6:3.5 mix, with 7¼ gal of water. Compared with the brick slab of equal dimensions the reinforced concrete slab deflected 0.585 in. at the ultimate load of 10 485 lb, while the brick slab deflected 0.390 in. at the ultimate load of 10 985 lb. They both failed in shear at about the same stress, 150 lb per sq in. for the brick, and 147 lb per sq in. for the concrete. This is not offered as a typical comparison, but merely as a matter of interest.

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[23] *The Clay Worker*, August, 1931, p. 82.

The following observations are of interest in connection with Professor MacCarthy's tests: The brick used had a compressive strength flat of 3 850 lb per sq in. Tested on end the compressive strength was 1 850 lb per sq in. The plain bars used for reinforcement had an elastic limit of 40 000 lb per sq in. at an ultimate strength of 73 200 lb per sq in. The specimens were tested at 28 days: 2 by 4-in. cylinders proved to have a compressive strength of 2 600 lb per sq in. All brick was thoroughly wetted before laying.

Items Nos. 21 to 25, Table 1, represent beams having an over-all length of 10 ft and a clear span of 8 ft. The distance between the concentrated loads was 2 ft 8 in. The age at which the beams were tested was 28 days. In Item No. 25 ¼-in. round bars, 8 in. on center, were used for stirrups and in Item No. 24 the stirrups were of No. 7 wire, spaced 8 in. on center. The other test specimens had no stirrups. The longitudinal reinforcement bars were neither hooked nor deformed. As indicated in the diagrams in Column (2), Table 1, Item No. 25 was designed with four bars, and the remainder with two bars. In Items Nos. 22 and 25, the bars were square instead of round as in the other tests; ¾-in. rods were used in Items Nos. 21 and 24; two of the rods in Item No. 23 were ½ in., and the remainder of the rods were ⅝ in., in diameter.

The features of Professor MacCarthy's tests were the use of a ⅜-in. mortar pargeting on the top and bottom of the slab and mortar joints so wide that the thickness between the rod and brick was at least one-half the diameter of the rod. The results obtained were quite satisfactory, but due to the use of the mortar pargeting no comparisons with other tests can be made. The five tests, Items Nos. 21 to 25, are likewise unusual in that a bottom course of mortar was used to embed the rods and a course of mortar for a finish; but it should be noted that the highest stresses were developed in Item No. 23 in which the rods were placed in the vertical mortar joint. The extent to which the mortar courses affected the results cannot be decided, because no check tests were made; but when the results in Table 1 are compared with other tests, it appears that the added care and trouble is not justified.

Items Nos. 26 to 31, Table 1.—Beams tested by Mr. Hugo Filippi, Engineer of the Common Brick Manufacturers Association of America, were constructed under conditions comparable to job practice. The six beams in Items No. 26 to 31, Table 1, were each 11 ft 3½ in. long over all and had a clear span of 10 ft. The distance between the concentrated loads was 3 ft 4 in. Three of the beams, Items Nos. 26, 27, and 28, were made of 1:1:6: mortar, and the remainder of cement-lime mortar in the ratio of 1:0.15:3. They were tested at ages of 23, 26, 28, 22, 25, and 27 days, respectively. These beams had no stirrups and the longitudinal rods comprised three straight bars $\frac{3}{8}$ in. in diameter, deformed. All the brick were salmons and culls with a compressive strength of 2 626 lb per sq in. They were constructed in freezing weather with no protection, the brick being sprinkled in the pile. The load was applied by piling brick on a platform supported at the third point of the beam. Failure of all beams was caused by shear, and, to quote Mr. Filippi, the beams "ultimately failed in bond between brick and mortar", with the consequent slipping of the bars after the diagonal crack opened. At the first crack, the average shear is calculated to be 25 lb per sq in. and, at ultimate loads, 40 lb per sq in. This early failure in shear limited the brick and steel stresses to values far below those possible, which indicates the necessity for stirrups or bent-up bars. The deflections were regular as the load increased and within the design loads they were much less than the allowable $\frac{1}{360}$ of the span length.

All the beams used in this test were constructed so that there was one course of brick below the steel. It is of interest that in no case did these lower bricks fall off.

Items Nos. 32 to 43, Table 1.—The slabs shown as Items Nos 32 to 43 were also built and tested by Mr. Filippi at the same time as the beams. He reports that in some of the slabs the mortar was frozen, and this explains the small loads carried in some cases. Six of the slabs, Items Nos. 32 to 37, had 5 ft clear span, and the remaining six had clear spans of 8 ft. The mortar mix for six of these slabs, Items Nos. 35, 36, 37, 41, 42, and 43, was 1:0.15:3. For the remaining six, the mortar mix was 1:1:6. The slabs were tested at ages varying from 18 to 24 days. Six straight, plain bars were used for reinforcement in each case. The slabs in Items Nos. 32 to 37 were $\frac{1}{4}$-in. round, and the remainder were $\frac{3}{8}$-in round. As in the beam test by Mr. Filippi all the bricks in the slabs were salmons and culls, with a compressive strength of 2 626 lb per sq in. All the slabs were built in freezing weather, with no protection.

In general, the results were as might be expected and indicate that, for spans of 5 ft, a $2\frac{1}{4}$-in. brick slab is safe for loads up to 50 lb per sq ft, and that $3\frac{3}{4}$-in. slabs can be designed to carry 60 lb per sq ft on spans of 8 ft, with a factor of safety of from 5 to 6. In all but two cases (Items Nos 39 and 42), failure was gradual, with regular increases in deflection as the load increased. Two of the slabs were loaded to three times the designed amount and when it was removed, full recovery of deflection was made, showing once more the capacity of this type of construction to recover its deflection. Fig. 2 shows Item No. 43 during the test, carrying 397 lb per sq ft.

Items Nos. 44, 45, and 46, Table 1.—These specimens were built and tested by the writer under the inspection of the Building Department of the City of New York. It was the intention to build them under conditions secured, and with the speed usually obtained, on a contract job. However, these conditions were exaggerated in the case of Item No. 44 when sand for the last batch of mortar had to be scraped from the ground, resulting in a dirty mortar which was used in the top course at the center of the beam. This no doubt contributed to the early failure in compression obtained on this beam.

The two beams had 16-ft and 11-ft spans, respectively, and the slab an 8-ft span. The mortar was 1:0.25:3 cement lime; and the brick, common clay with an average compressive strength of 3 044 lb per sq in. All work was done in the open, and wetted for 10 days. Testing was done at the age of 30 days with improvised platforms resting at the third points on all items. Deformed bars with straight hooks and double $\frac{1}{4}$-in. stirrups were used. Bending was done at the site. Item No. 44 was built in place; Items Nos. 45 and 46 were built on the ground and lifted to the supports on the day of the tests. In doing this several courses were broken from the side of the slab, thus reducing its width by 8 in. to 30 in.

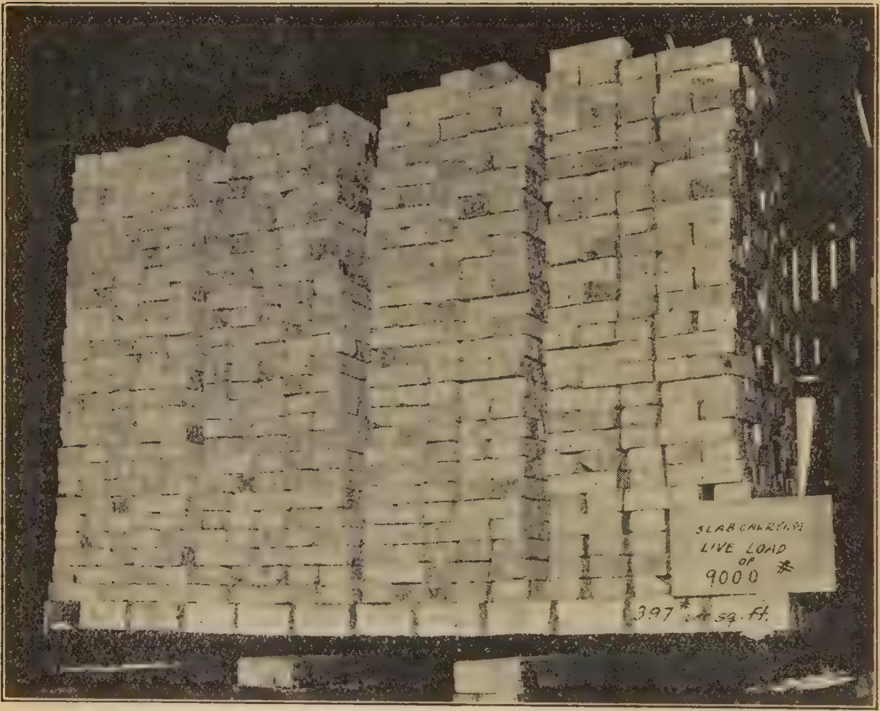

FIG. 2.—SLAB ONLY ONE BRICK THICK SUPPORTING 397 LB PER SQ FT; LIVE LOAD OF 9 000 LB (SEE ITEM NO. 43, TABLE 1)

Both beams gave away suddenly in the manner of a breaking stick at loads equal to 2.4 (Item No. 44) and 4.8 (Item No. 45) times the designed loads. The first crack and allowable deflection were at loads slightly greater than twice the designed load in each case. Although the early failure in compression limited the development of shear stresses to 144 lb per sq in. (Item No. 45), still the absence of any indication of shear cracks shows that this stress can be taken care of, even though mortar joints pass completely through the beam every 8 in. as in these items. The tension in the steel (Item No. 45) was 49 500 lb per sq in. at ultimate load. Considering the dead load also, this stress is computed as 64 500 lb per sq in. Similar increases apply to all the other beams.

These failures were unusual, in that the buckling action at the center caused the top course to pull away for a length of 30 in., causing a complete destruction of the section directly beneath it, although the remainder of the beam was intact.

The slab (Item No. 46) failed gradually. The test was discontinued because of excessive deflection at a load equal to 4.4 times the design load of 60 lb per sq ft, or 900 lb on the basis of third-point loading. After the load was removed five men stood on the slab, and it seemed as stable as ever even if greatly bowed.

The writer understands that other tests are being conducted which indicate that the value of n for reinforced brick masonry should be greater than that assumed herein. However, as nominal variations in the value of n do not materially affect the shear stresses, it is not of great importance, as a review of the stresses developed by the masonry in compression and the known ability of steel to take high tensile stresses give assurance that there is an ample factor of safety even if the exact distribution of stresses is not known.

If future investigations indicate that $n = 30$, the values given herein for the brick stresses will be reduced by about 16%, the steel stresses increased by about 4%, and the shear stresses decreased 4 per cent. When working stresses are based on a factor of safety of 3, or more, these possible errors are unimportant.

Several other tests have been conducted, but due to the incomplete reports available, no definite conclusions can be drawn from them. For instance, Major Lent tested $2\frac{1}{4}$-in. and $3\frac{3}{4}$-in. slabs reinforced with light-weight, expanded metal, diamond mesh. That is, the bricks were laid in a bed of mortar on the mesh and tested on a 4-ft span, which required a load of 520 lb per sq ft to produce a deflection of $\frac{1}{8}$ in., which is $\frac{1}{360}$ of the span length, and made practically a full recovery when the load was removed.

Of course, such variables as the strength of brick and mortar have considerable effect on the capacity of the beams and slabs, but due to the wide variation in all elements of the structures tested in the experiments discussed herein, and to the fact that the tests in general were not designed to determine single characteristics, it is difficult to draw conclusions as to the extent of their influence.

Reviewing Table 1, therefore, it is found that: (1) Deflections in reinforced brick masonry beams and slabs are consistent and small within working loads (see Fig. 3) (usually they are much less than the allowable deflection of $\frac{1}{360}$ of the span length); (2) upon removal of the load, recovery of deflection, sometimes 100%, is possible; (3) granting the assumptions on which the calculations are based, rods inserted in mortar joints will develop bond stresses as high as 384 lb per sq in. (see Item No. 8), with an average of 229 lb per sq in. when rods are placed in vertical joints (see Table 2); (4) shear stresses are affected by the manner in which the bricks are placed, and by proper design can be developed to as high as 154 lb per sq in. (Item No. 8) without shear reinforcement; (5) the introduction of stirrups increases the beam's resistance to shear (see Table 2); (6) when properly designed, the tensile strength of longitudinal reinforcement can be fully developed; and (7) the compressive strength of the masonry in beam action is greater than the compressive strength of wall panels in direct compression. Table 2 does not take into consideration the manner of failure. Therefore, one or the other stress is always less than that possible; thus, the values listed offer conservative values.

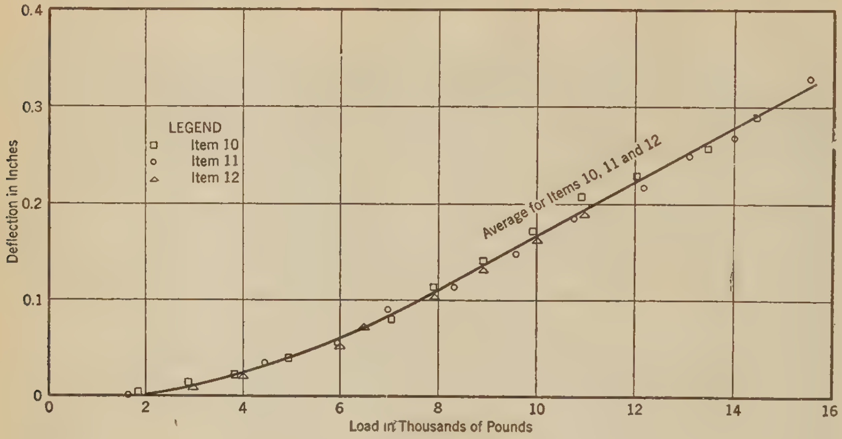

FIG. 3.

TABLE 2—AVERAGE AND MINIMUM UNIT BOND STRESS AND SHEARS

Item No. (1)	Description (2)	UNIT STRESS, IN POUNDS PER SQUARE INCH: Bond: Number of beams averaged (3)	Bond: Stress: Average (4)	Bond: Stress: Minimum (5)	Shear: Number of beams averaged (6)	Shear: Stress: Average (7)	Shear: Stress: Minimum (8)
1	All beams listed at first indication of failure	31	124	51	31	61	23
2	All beams listed at ultimate load	32	187	65	32	88	29
3	Rods in vertical joints at ultimate load	18	229	82	.....	....	.....
4	Rods in horizontal joints at ultimate load	9	116	65	.....	....	.....
5	Rods in mortar at ultimate load	4	164	98	.....	....	.....
6	All beams without stirrups at first indication of failure	.....	....	....	24	53	23
7	All beams without stirrups at ultimate load	.....	....	....	25	83	29
8	All beams with stirrups at first indication of failure	...	....	....	7	90	61
9	All beams at ultimate load with stirrups	.....	....	....	7	108	68

CONCLUSIONS

In view of the evidence submitted in this paper, the writer presents twelve conclusions for discussion, as follows:

1.—The assumptions accepted in the design of reinforced concrete structures can be used in the design of reinforced brick masonry items similar to those discussed herein.

2.—The recommended working stress of 0.4 f'_c given by the Joint Committee on Standard Specifications for Reinforced Concrete can be safely allowed on reinforced brick masonry. This would allow approximately 750 lb for cement-mortar brick masonry and 500 lb when cement-lime mortar is used, based on tests of full-sized wall panels.

3.—A test of piers made up of two or three brick-and-mortar joints would give better results on which to base such working stresses and no doubt would increase the foregoing values by about 10%, as in the case of concrete cylinders and concrete walls. When this is done, the limiting values would be roughly 800 lb per sq in. and 550 lb per sq in.

4.—The use of smaller bars ($\frac{1}{2}$-in. round) is more feasible and will result in higher bond strength.

5.—Bond stresses should be limited to 80 lb per sq in.

6.—Shear stresses should be limited to 25 or 30 lb per sq in., which would necessitate stirrups or bent-up bars in most beams.

7.—To develop maximum strength, the reinforcing rods should be placed in vertical joints.

8.—The face of the brick that develops the greatest compressive strength should be placed normal to the line of compressive stress in the beam.

9.—The value of the ratio, $\frac{E_s}{E_b}$, is somewhere between 20 and 30.

10.—The mortar should consist of 1 part cement, 0.25 part lime, and 3 parts sand, by volume.

11.—Builders should adhere rigidly to all the requirements of good brick masonry; that is, the proper wetting of the brick and complete filling of joints.

12.—Further well-directed study and testing will undoubtedly justify an increase in the allowable stresses.

Given these conditions, this type of construction can be used with safety when designed by the formulas given in the report of the Joint Committee on Standard Specifications for Reinforced Concrete. The development of any system of construction is governed by economic considerations, and there are certain handicaps to the use of this type, such as (*a*) the necessity of placing rods in mortar joints, which will sometimes result in an excess amount of steel; (*b*) the need for considerable hand labor, both of which, (*a*) and (*b*), are expensive; and (*c*) the difficulty of securing water-tight joints in exposed work to prevent the corrosion of the steel. There are undoubted advantages, however, such as the elimination of all vertical form work, the flexibility of operations permitted in a small working space, the finished appearance, and the invisibility of cracks—which will often more than balance the disadvantages for such construction as retaining walls, circular tanks (see Fig. 4), piers, and bridge bents, and floor-slabs when architectural considerations require them.

Other possibilities that suggest themselves are stadiums, foundation walls, tunnels, etc., where form work would be intricate and expensive, or concrete difficult to place. Reinforced brick curtain walls have been utilized[24] in a six-story steel frame structure with 8-in. walls, in San Francisco, Calif. This use of reinforced brick masonry would appear to be logical in areas subject to earthquakes. Then, of course, there are situations in which concrete and steel are not available, when a knowledge of this method of construction would be of great value. The weight of such construction is about 20% less than that of concrete, which would have considerable effect in a multiple-story structure, and the dead load can be further decreased by the elimination of ceiling and wall plaster in buildings where the structural members will become decorative when made of brick. The use of brick for interior finishes is found in ancient as well as extremely modern buildings and the variation in color obtainable in ordinary brick allows the designer great scope for decorative effects.

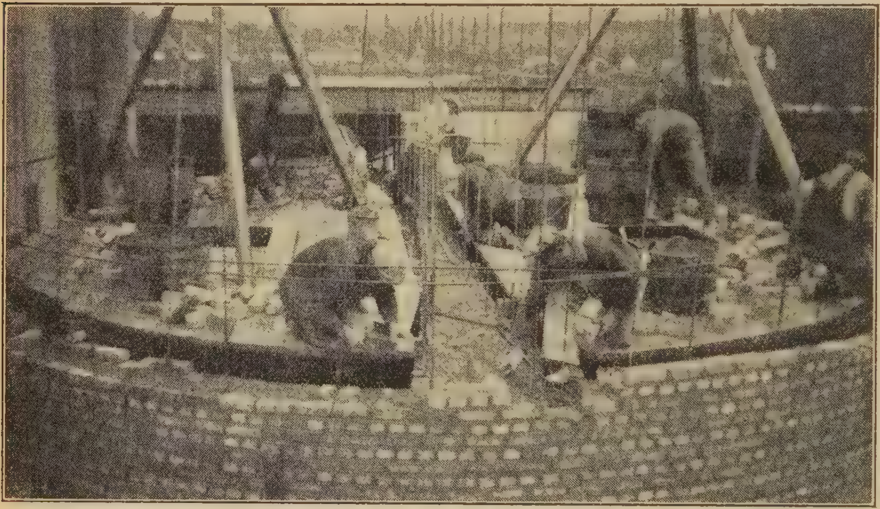

FIG. 4.—STORAGE BINS AT WENDRON, ILL., SHOWING REINFORCEMENT.

If developments indicate that specially shaped brick would be advantageous, such as slotted or grooved brick, the brick industry will no doubt respond as it did when radial brick for chimneys and feather brick for arches were deemed necessary.

ACKNOWLEDGMENTS

The writer is indebted to Carl T. Schwarze, M. Am. Soc. C. E., for offering helpful suggestions after reading this paper. The paper was prepared from data secured while the writer was employed by the New York and New Jersey Common Brick Association, and acknowledgment is due that source.

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[24] *Brick*, Vol. 43, No. 6, 1913.

DISCUSSION

W. K. Hatt,[25] M. Am. Soc. C. E. (by letter).—Under the heading, "Characteristics of Brick and Brick Masonry," Mr. Hansen states that the Building Code Committee of the U. S. Department of Commerce has recommended 600 lb per sq in. as the allowable working stress in direct compression on the best brick masonry. The Committee gives a more complete definition of the circumstances in which the high stress of 600 lb per sq in. may be applied; for example, in Section 4, under the heading, "Working Stresses,"[12] the Committee stipulates that "the allowable compressive stresses in brick masonry due to continued live and dead loads" shall not exceed the limits given in Table 3. An exception is made in the case of masonry "laid with smooth level horizontal joints and completely filled vertical joints."

TABLE 3.—Allowable Working Stresses

Grade of brick; average minimum compressive strength tested flat, in pounds per square inch	Allowable Working Stresses on Gross Cross-Sectional Areas, in Pounds per Square Inch:		
	Lime mortar	Cement-lime or natural cement mortar	Portland cement mortar
8 000+	100	300	400
4 500 to 8 000	100	200	250
2 500 to 4 500	75	140	175
1 500 to 2 500	50	100	125

If such joints are thoroughly inspected, and if "the effects of eccentric and concentrated loads and lateral forces" are fully considered, the working stresses in Table 3 may be increased by 50 per cent. When masonry is composed of brick of different grades the allowable working stress is required to be that listed for the lowest grade of brick of which the masonry is composed.

C. T. Schwarze,[26] M. Am. Soc. C. E. (by letter).—A future of usefulness for reinforced brick masonry will depend upon normal resumption of construction work. Until business and finance have regained normal activity whatever of science is involved in this revived method of construction should be persistently, although carefully, worked out through research. That there is the possibility of future usefulness for reinforced brick masonry is evident. However, to make such a future possible, a careful study of the behavior of brick masonry in combination with steel and the use of steel to overcome the patent weakness of brick masonry in tension must be made. The profession must know what modification of formulas, proposed in the report of the Joint Committee on Standard Specifications for Reinforced Concrete,[22] must be made to undertake the design of reinforced brick masonry for construction members.

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[25] Prof. of Civ. Eng., School of Civ. Eng., Purdue Univ., Lafayette, Ind.

[26] Prof. of Civ. Eng., New York Univ., New York, N. Y.

Mr. Hansen has called attention to joint action of steel reinforcement and brick masonry. That the author is justified in recommending the use of construction formulas, or modifications of them, advanced by the Joint Committee on Standard Specifications for Reinforced Concrete, becomes evident after a study of test data. To justify this use, longitudinal joints, where horizontal steel is placed, must be made wide enough to embed the reinforcement completely without endangering the cohesion of the brick masonry.

Professor MacCarthy's tests (Table 1, Items Nos. 21 to 25) indicate that a space of mortar between brick and rod of one-half the rod's diameter would be ample for the development of full bond strength for larger rods. For smaller rods a full diameter on each side would probably be a wise precaution. Stirrups should also be embedded throughout their full length to take, completely, vertical components of diagonal tension through bond.

Of real interest is the comparative tests made by Professor MacCarthy between brick masonry and concrete slabs. It is to be regretted that only one mortar slab and one concrete slab were fabricated and tested. It is suggested as a matter for future research that more comparative tests between reinforced concrete and reinforced brick masonry be made under the same conditions of design and loading. It should be noted that, in the comparative tests, the reinforced brick masonry slab showed considerably more stiffness than the concrete slab. For practically the same ultimate load of between 10 500 lb and 11 000 lb the brick masonry slab deflected less than two-thirds as much as the concrete slab.

Attention is called to this apparently greater stiffness of brick masonry since it verified observations made by the writer during tests of reinforced brick masonry beams for the Department of Buildings, City of New York, under the auspices of Mr. Hansen (Table 1, Items Nos. 44 to 46). The condition under which these beams were loaded was very unusual. Large platforms had been built to bring the load to the third points of the beams. Loading was effected by building up bricks in interlocking layers by workmen, several of whom were on the platforms placing the bricks. As these men moved about on the platform, the beams rocked back and forth on their bearings. Deflection readings could only be obtained after the men had left the platforms. One of the officials of the Building Department remarked, "What these beams are getting is live load with real impact!" The rocking was much worse for Item No. 44 than for Item No. 45 which may partly account for failure in the former at 2.4 times the design load as compared with 4.8 times the design load in the latter.

During this test the writer was impressed with the behavior of the beams in compression. When failure occurred in Item No. 44 (Table 1) on the compression side, it did so at a point where inferior sand had been used in the mortar, but with a suddenness that was startling. Up to the moment of rupture, the beams exhibited the rigidity in compression noted previously in analyzing Professor MacCarthy's results. This rigidity indicates a factor in favor of reinforced brick masonry over concrete, which is herewith presented as a stimulus to further research and verification.

Many new and astounding facts regarding the behavior of concrete have come to light in recent years. Probably the most serious of these is that of plastic flow. How seriously this may affect the design of compression members, such as columns, is evident from the results of experiments made at Lehigh University and the University of Illinois, as reported by Committee 105, on Reinforced Concrete Column Investigation, of the American Concrete Institute,[27] as follows: "If the load is sustained for some time, the stress distribution changes very rapidly due to plastic yielding and volume of concrete." Approximately from the time when concrete came into general use it became known that a "permanent set" occurred when compression loads were applied. The compression side of a reinforced masonry beam is composed of from 88 to 90%, hard burned argillaceous material throughout its length, which has no plastic flow. Only from 10 to 15% is of cementing material, as compared with 100% in a concrete beam. Apparently, this accounts for the rigidity in compression and low deflection in brick masonry beams. Such beams as Items Nos. 44 and 45 (Table 1) failed by shattering brick along the top courses. Mr. Hansen's words, "gave away suddenly in the manner of a breaking stick," about describes such a failure. More research along this line is needed before definite conclusions may be reached, but it does appear as if the rigidity in compression for such beams was governed by brittleness in the brick when reinforcing steel had been stressed beyond its yield point. This idea is in accord with Mr. Hansen's Conclusions 1 and 2 in a review of Table 1.

The author of this interesting paper makes suggestions as to possible uses for, and advantages of, this type of construction. The beauty and durability of brick are legendary. Earliest uses, from archæological discoveries, were in the Valley of the Euphrates about 5 000 years ago. Since that time brick masonry has given unbroken and faithful service to Man in nearly all parts of the globe. About a century ago, apparently, the first use of brick masonry, with steel as an aid, came into being. Present-day research has shown almost limitless possibilities for this combination of two time-honored materials of construction. As Mr. Hansen points out, these possibilities are both esthetic and utilitarian, and it is to be hoped that the present stimulus in research will demonstrate both feasibility and economy in construction work for reinforced brick masonry.

J. W. McBurney,[28] Esq. (by letter).—The author has performed a service in making generally available, and in correlating, certain hitherto unpublished data on reinforced brick masonry tests.

Mr. Hansen gives averages for the ratio of compressive strength of brick flatwise to compressive strength on edge, modulus of rupture to compressive strength flatwise, etc. He states, "however, there is a considerable variation of these ratios for brick in different territories" (see heading, "Compressive Strength"). It may be of interest to know that the ratio of compressive strength flatwise to compressive strength on edge ranges from 0.74 to 2.30 and the ratio of modulus of rupture to compressive strength flatwise has a range of values from 0.070 to 0.426.[29] It should be emphasized that most of this variation in ratio is not due to chance or error. A particular type of brick is usually characterized by a rather constant and definite ratio. As the writer has stated previously,[30] the fact that a given ratio may be the average for a very large number of tests does not increase the probability that the average ratio will represent the actual ratio for a particular kind of brick.

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[27] *Journal*, Am. Concrete Inst., Vol. 4, No. 6, February, 1933, p. 277.

[28] With Bureau of Standards, U. S. Dept. of Commerce, Washington, D. C.

This same point of view should apply to reporting compressive tests on brick masonry. Referring to a paper of which the writer was a co-author, Mr. Hansen credits the Bureau of Standards with reporting that the average strength of wallettes, 18 by 34 in. high with 1:0.1:3 cement mortar, was 2 110 lb per sq in. To secure this average, he includes items given in the report that belong in separate categories, and, therefore, his statement is open to question.

The writer disagrees also with the statement that "it is generally conceded that walls 9 ft high will develop a compressive strength equal to from 25 to 30% of the brick strength." Considering brick strength, mortar, workmanship, and regularity of size and shape of the brick units as factors, the authors of *Research Paper No. 108* reported ultimate compressive strength of brick masonry expressed as percentages of brick strength, ranging from 8.8 to 44.0%, restricting the comparison to clay bricks laid as solid 8 and 12-in. walls.

It is possible that the man who runs as he reads may not understand the implications of "best brick masonry" referred to in the statement of the recommendation of the Department of Commerce Building Code Committee. The working stress of 600 lb per sq in. is limited to the following combination of factors: (1) The brick must be at least 8 000 lb per sq in. in compressive strength tested flatwise; (2) the mortar must be Portland cement mortar as defined by the Code; (3) the workmanship must be thoroughly inspected and must be such as to produce smooth, level, horizontal joints and completely filled vertical joints; and (4) the effects of eccentric and concentrated loads and lateral forces must be fully analyzed and allowance made for them. For certain other combinations of factors, the Committee limits the working stress to 50 lb per sq in.

It is suggested that mortars with a cement-lime-sand ratio of 1:0.15:3 be referred to as cement mortar with an addition of lime. There is considerable evidence that the strength of a 1:3 cement mortar is not impaired by the addition of not more than 0.25 parts of lime by volume. Therefore, the practice is recommended of limiting the term, cement-lime-mortar, to compositions ranging between 1:0.25:3 and 1:2:9, of which 1:1:6 is typical.

E. G. Walker,[31] M. Am. Soc. C. E. (by letter).—The author has condensed into the compass of a short paper an excellent summary of the recent American experimental work on reinforced brickwork. In tracing the historical development from the time of the first Brunel, he stops short at the 1851 Exhibition, and leaves the impression that nothing has been done in Great Britain since that time. In fact, the practice of reinforcing brickwork in an empirical manner by the use of flat iron strips or round iron rods has been in use ever since the early days described by the author. The use of the hoop-iron bonding, which consisted of strips of flat iron hoops laid continuously in the horizontal mortar beds at vertical intervals of a few courses, has been, and is, a well-known method of binding together brick structures. Reinforced brickwork with round rods has also been developed, and structures similar in character to that illustrated in Fig. 4 are not uncommon.

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[29] "The Compressive and Transverse Strength of Brick," by J. W. McBurney, *Research Paper No. 59*, National Bureau of Standards, p. 834.

[30] *Loc. cit.*, p. 833.

[31] (Maxted & Knott), London, England.

Contracting firms have specialized in this class of work. Although it is not correct to state that the art of reinforcing brickwork was forgotten, nevertheless, the broad conclusion is true that the development of reinforced concrete construction, which first began in Great Britain on systematic lines about 1898, has retarded a development in brick construction which probably would have taken place otherwise. The reinforced brickwork that has been built during the Twentieth Century has been more usually in the form of vertical walls, panels, and columns which, by being reinforced, have been made thinner. Of the more scientific use of the material for beam, slab, arch, and similar constructions of kinds common in reinforced concrete, there have not been many examples, even though the pioneering work was done in and around London by men famous in the annals of British Civil Engineering.

In India, conditions are quite different. Throughout Southern Asia the brick has been the pre-eminent basis of permanent construction from the earliest recorded times. British engineers in India, who must utilize local material and local labor for their work, have thus turned naturally to the possibilities of brickwork rather than concrete. Until about 1913, all cement used had to be imported into India. The well system of constructing foundations, so preponderatingly used in India, is an excellent example of the supremacy attained by brickwork there.

In countries such as the United States and Great Britain, brickwork does not start with any such advantages as it does in India. Indeed, the opposite is the case. Concrete-making materials are easily attainable in most parts of these countries; uniform high quality is general; and the machinery and technique of concrete making and placing are highly developed. Pouring concrete, on a job of any size except the smallest, is mainly a mechanical process involving little direct labor beyond the spreading, spading, and ramming of the concrete in the forms. Considerable hand labor must be employed, however, on many jobs, in making and preparing forms, and this is never entirely absent, even in structures in which the greatest possible use is made of steel forms and repetition details.

Against this, reinforced brickwork of the modern type, to which the author calls attention, requires little falsework beyond strutting and lagging of soffits or centers for arches. The main structural material, however—the brick—must be placed entirely by hand labor of an expensive kind. The brickwork structure, too, must be built with a vast number of joints, the positions and arrangements of which are determined beforehand by the dimensions of the brick, the system of bonding, and the shape of the completed structure. There is, therefore, much more limitation upon the disposition and permissible shape of the reinforcement than there is in the equivalent concrete structure.

These general considerations show that both systems of construction must have their spheres of usefulness among the range of practical conditions under which structures must be built. The development of reinforced brickwork was largely arrested during the second half of the Nineteenth Century because of lack of scientific knowledge of design, and during the earlier years of the Twentieth Century by the continuous improvement in the manufacture of cement. The technique of preparing concreting materials, of making concrete, and of scientific designing of concrete structures, has been greatly advanced. Now there is no reason why reinforced brickwork should not take a recognized place among structural materials. In the days of the Brunels and the other pioneers of reinforced brickwork, hand-made bricks only were available. The modern machine-made brick is a far superior product in strength, shape, and uniformity of quality; it conforms to much closer limits of dimension and is in every way a more dependable product. It remains only to put the design of reinforced brickwork on a sure basis, so that engineers may have available an alternative structural material, which they will be able to use with advantage when economic conditions are suitable.

All those who have experimented with reinforced brickwork are agreed that the same fundamental bases of design may be used as are generally accepted for reinforced concrete. There is no reason for disputing this conclusion, since the fundamental conditions must be the same in both cases. The flexibility, which is provided in a reinforced concrete beam by the fact that the concrete will resist small tensile stresses and thus accommodate itself to the development of larger tensile stresses in the reinforcement, is provided in the brick beam by the mortar of the joints. Concrete and brickwork are equally suitable for compressive loading. Conditions for the development of bond stresses should be the same in brickwork as in concrete if adequate sectional area of mortar is provided around the reinforcement and if attention is paid to the proper filling of all joints in which steel is embedded. Provision of shear reinforcement is perhaps more difficult, since in its arrangement attention must be given to the disposition of the individual bricks. The limitations that result from this may also curtail the freedom of the designer to use the reinforcement to the greatest advantage.

No mention is made in the paper of the effect of brick bond on the strength of reinforced brickwork. It seems reasonable to suppose that variations in bonding must have their effects on strength, particularly on shear strength. This is a feature in which direct comparison is not possible with reinforced concrete, where for design purposes the concrete section may be taken as uniform. Vertical sections of a brick beam change suddenly in character, according as they are taken through vertical joints or not, and the arrangements and disposition of the vertical mortar joints must vary according to the bond adopted in building the beam. It seems likely that this variability of conditions has its effect on the distribution of shearing resistance and may account in some measure for the large variations of unit bond and shear stresses shown in the author's Table 2.

The summarization of experimental results given in the paper shows that there are very wide ranges of ultimate strength. On the whole, these ranges are much wider than are found in modern researches on reinforced concrete. While it is possible to take from them reasonable figures to form a basis of design, it certainly seems that considerable further co-ordinated research is necessary in order to put design on a sure and economic basis. Mr. Hansen mentions cases in which the full value of experimental results cannot be attained, because of uncertainties in the conditions under which the observations were made. In reinforced brickwork the distributions of stresses are probably more complicated than they are in reinforced concrete. When the volumes of research on the two materials are compared, one is driven to the conclusion that there is a large field still open for co-ordinated investigation before generally accepted rules of design for reinforced brickwork can be laid down with the degree of certainty necessary for its full development as a structural material.

JAMES H. HANSEN,[32] JUN. AM. SOC. C. E. (by letter).—It has been suggested by Messrs. Hatt and McBurney that the writer should have given, in more detail, the qualifications pertaining to the various recommended values for brick masonry and brick strengths. Their criticisms are legitimate and should be taken into consideration by any one whose purpose it is to develop the primary assumptive values to be used in the design of reinforced brick masonry. The object of the paper, however, was to give a short, intelligible résumé of what had been done. Obviously, the variations in all types of brick and brick masonry through geographic location or workmanship could not be discussed without involving a mass of detail from which it would be difficult to derive conclusions. Furthermore, the qualifications pertaining to the recommended allowable working stress of 600 lb per sq in. for brick masonry, which Professor Hatt states should be given, are of such a nature as to be vague and almost indeterminable. For instance, what constitutes a "thoroughly inspected joint," and when are the "effects of eccentric and concentrated loads and lateral forces" fully considered? Such qualifications are commendable cautions, and serve to indicate that brick masonry of such a nature can be (and often is) obtained, and that, therefore, it should be given legitimate consideration in the development of reinforced brick masonry.

Tests conducted by Inge M. Lyse, M. Am. Soc. C. E., under the sponsorship of Mr. Judson Vogdes substantiate that opinion. Professor Lyse reports tests[33] on fifteen columns, in which average solid brick strengths were 13 760 lb per sq in. flat, 10 680 lb per sq in. on edge, and 10 070 lb per sq in. on end. The columns were $12\frac{1}{2}$ in. square and 10 ft high. The maximum loading with cement-mortar and no reinforcement in one case was 800 000 lb, or about 5 120 lb per sq in. Furthermore, Mr. Hugo Filippi reports the results of a large testing program[34] in which plain brick piers, approximately 8 by 8 by 17 in., took average maximum loads of 2 296 lb per sq in.

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[32] Executive Secy., Brick Mfrs. Assoc. of New York, New York, N. Y.

[33] *Engineering News-Record,* March 16, 1933.

Professor Schwarze has suggested that plastic flow in brick masonry is less than in concrete. No long-time load tests have been conducted, to the writer's knowledge, to establish this contention. However, it does seem probable that such is the case and that the distribution of stresses based on the values of n, as customarily established, will result in more accurate stress determinations in brick masonry than in concrete. This may prove a decided advantage for this type of construction. In correspondence with the writer, Professor Schwarze states that recent tests under his supervision have strengthened his belief on this point. These tests on six piers, 8 in. square by 30 in. high, reinforced with $\frac{1}{2}$-in. round rods and $\frac{1}{8}$-in. ties, 6 in. on center, revealed no evidence of plastic flow and gave results of 1 500 lb per sq in. in compression when rusted rods were used. Failure was caused by crushing of the brick, and there was no indication of bond failure. The value of n for column design was established as 12.

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[34] *Brick Engineering*, Vol. III.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1873

THE PLAN OF BOSTON, MASSACHUSETTS: A CAPITAL CITY[1]

By Arthur C. Comey,[2] M. Am. Soc. C. E.

With Discussion by Robert Whitten, Esq.

Synopsis

State capitals, by example, inspire other cities in planning. The State functions may dominate a small city, or they may merely furnish the basis for a fine architectural treatment, as in Boston, Mass., the largest State capital. Boston's plan relates to its commercial and other municipal activities as a metropolis. It also shows the effect of its long history of 300 years. It still has many early architectural gems.

At first, the streets were narrow and crooked. Later, checker-board streets were laid out in new territory and on filled-in mud-flats, thus, destroying Boston's peninsular shape. Some streets were made wide only to become narrow again upon entering the still more recent suburban districts, many of which became a hodge-podge of narrow streets leading nowhere.

Since about 1890 broad planning has received more and more attention. After a long series of haphazard street widenings, an official Planning Board prepared a major street plan in 1930. This Board had previously zoned the city and has become recognized as the central authority on planning matters.

Waterways, long ignored, have become the centers of beautiful parkways. Since the creation of the Board, Boston's port development has followed a general plan. For some time railroads have been fixed, with two consolidated terminals. Rapid transit, beginning with the first American subway, taps all inner suburban areas. Boston's park system has led the way; likewise, the City established the first municipal playground in the United States.

Boston has no civic center, but the State House faces the great central Common. It is protected by a low height limit imposed on adjoining build-

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[1] Presented at the meeting of the City Planning Division, Boston, Mass., October 10, 1929.

[2] Cons. City Planning Engr., Cambridge, Mass.

ings. Boston's pioneer control of building heights by districts was followed by comprehensive zoning. By statute dated March 19, 1928, pyramidal buildings with the same volume as a solid 155-ft building are permitted.

As Metropolitan Boston includes thirty-nine or more municipalities there was little regard for the greater city's future prior to 1929. Only water supply, sewage disposal, and the larger park features were under the control of Metropolitan Boards, which were merged into a single commission in 1920. The State exercises much detailed authority over the area. It has set up a Division of Metropolitan Planning, which has prepared a thoroughfare and transit plan as a guide for the Legislature and the individual communities. The State Department of Public Works also has prepared plans both within and without the City of Boston. While there is overlapping, there is no metropolitan planning of zoning, and little co-ordination of many other factors in a regional plan. The Division of Metropolitan Planning probably should be in charge of such a plan covering all problems affecting more than one municipality.

The chief justification of a paper such as this lies in the inspiration and guidance that the capital city may exert through its example on other cities within the State. The prototype in this is the Capital of the United States, which, itself originally well planned and now receiving renewed attention as to its planning, undoubtedly influences profoundly the quality and extent of planning in cities throughout the country. State capitals can seldom if ever approach in effect that of the Federal Capital, but they may, and do, in many instances, lead the State in city planning ideals.

Boston, the Largest Capital

A second factor worth comparing in capital cities is the relation of the State's administrative function to the city's physical plan. This also varies exceedingly in importance, largely according to the magnitude of each city's remaining functions. Denver, Colo., presents what may be considered the arch-type.[3] Another type is that represented by Olympia, Wash., in which the population is small and the capital is devoted almost entirely to State administrative activities. In such a capital as Boston—the largest of them all — the presence of the State Government is only an incident. To be sure, the one large capitol building is the crowning jewel of an architectural composition in one part of the city (see Fig. 1), but otherwise it exercises practically no effect upon the city plan. To satisfy oneself that this is so, it is only necessary to compare the elements of Boston's city plan with the same elements in any other great center of population. The city is one of the small group of world metropolises. Its effect upon the surrounding region is due to its size, to its being the commercial center of this region, to its being a seaport, and to the industrial, educational, and other activities that have been attracted to it. Had the capital been placed, as in many States, in some small more central city, Boston's plan would have shown few or no differences. Therefore, this study is directed at Boston as a metropolitan commercial city and not as a capital.

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[3] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1312.

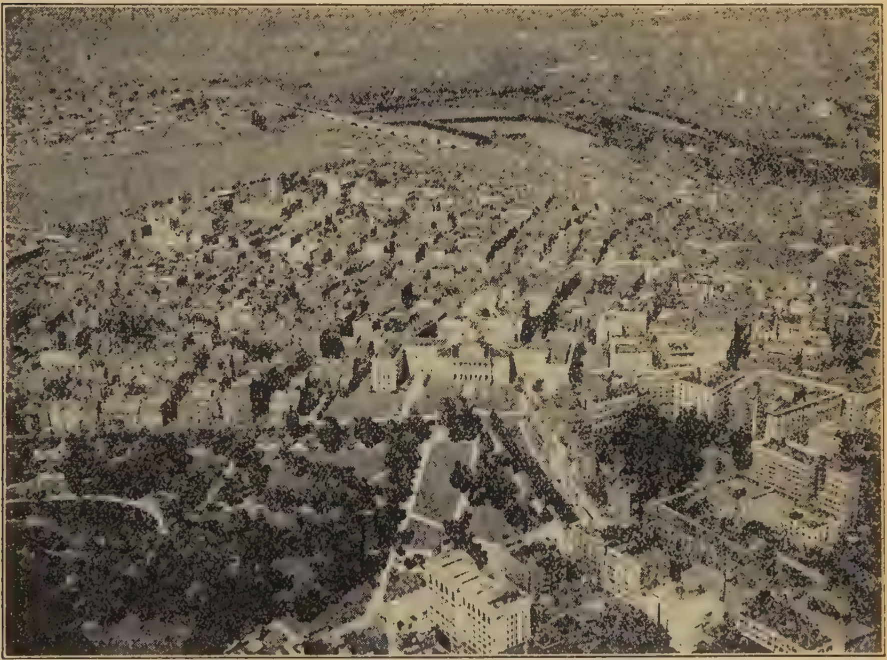

FIG. 1—THE STATE HOUSE ON BEACON HILL, BOSTON, MASS.

ORIGIN AND EARLY DEVELOPMENT

Boston differs also from most other capitals, particularly those of the West, in being one of the oldest cities in the United States, and thus exhibiting much more of an historical character in its plan. The tercentenary of the city was celebrated in 1930. Never planned in any large degree, it has grown steadily from the small Seventeenth Century Colonial village and carries in its layout many visible evidences of each successive era. The city's history is a subject that is worthy of extended study by the city planner. The first settlement was across the Charles River from Boston proper, at Charlestown, which was laid out by an English engineer, Thomas Graves, with 2-acre lots for each inhabitant. In Colonial days, Boston was situated between and upon three hills on a peninsula with a long, narrow neck, easily defended from the Indians. It was flanked by salt marshes, but jutted into the deep water of a fine harbor. On one side land was reserved for a common ground, and the Boston Common still remains as the beginning of the city's park system. In this first era the streets were nearly all narrow and often crooked, serving the immediate need only. Many gems of architecture, both public and private, remain from this and the immediately succeeding periods, but few of the other municipal activities have left a permanent stamp on the present city plan.

However, at an early date, mud-flats and other areas began to be filled and this work has continued spasmodically, resulting in great changes in the topography. First, certain coves were obliterated; then large areas along the principal estuaries were filled until, to-day, Boston is no longer a peninsula; it is even losing its shape as a broad point. Each of these filled areas offered a comparatively clean slate for a new street plan and—as in the case of practically every other planned city from the earliest Egyptian and even lacustrine times practically to the present century—these fragments were planned on a checker-board system, or as close to it as their marginal limitations permitted.

At this same period, Boston began to extend its city limits, absorbing neighboring towns and intervening land in several directions although not in all. Where growth was slow these towns expanded in a haphazard fashion, but certain large areas that came into the market about 100 years ago shared in the world-wide attempt at a grander scale of city building. These areas were laid out with straight streets (some of them very broad) again plotted in the checker-board plan, in some cases practically regardless of unfavorable topography. Thus, the street plans of East Boston, South Boston, the South End so-called, lying between the old Boston proper and Roxbury, and, later, the Back Bay District, were added, with a resulting ease of movement of traffic, the benefits of which are being reaped to-day.

During this early Victorian period even the pre-existing roads were widened in some cases as they passed through the newly planned belt, so that Washington Street, for example, broadens from 50 ft to 100 ft, and more, in width in the South End only to become narrow again at its entrance to Roxbury.

What may be considered the first city plan for Boston was developed from 1843 to 1845 by Robert Fleming Gourlay and submitted by him to the authorities as "Studies to Illustrate the Science of City Building"; but although he anticipated later needs with surprising accuracy, his plans apparently fell on deaf ears.

Later, the generous ideals of the early developers were abandoned, and many miles of streets were added to the city plan with little or no thought for community needs, so that the outer sections of the city are for the most part a hodge-podge of unrelated fragments—narrow streets leading nowhere. The pre-existing main streets usually passed through these suburbs with their original narrow country-road widths. However, here and there, a better plan was followed, already a few high-grade sub-divisions were being platted, and since the land was cheap in the outer circle, some of the thoroughfares again were widened. Thus, in passing outward on Blue Hill Avenue, after more than a mile of 60-ft street, the width changes to 120 ft from that point to the city line.

Development in Recent Years

Since about 1890 two special influences have been brought to bear on the street plan. The park system became important. To-day, the parkways laid out at that time are practically integral portions of the traffic network, although happily restricted to passenger travel.[4] Finally, broad planning, at least so far as related to streets, was attempted through a Board of Survey. Much of the unplatted portion of Boston was laid out on a general plan of wider streets, with a reasonable number of through connections and some relation to topography. Although the invocation of the police power to assure the completion of this plan was soon declared unconstitutional, many portions of the layout were accepted and now exist on the ground. While mechanical in some respects and not as fully related to the needs of the city as a whole as modern city planning would dictate, this street planning was a long step ahead and its methods have never been entirely abandoned by the Board's successor, the uneuphoniously but accurately styled "Street Laying-Out Department" (see Fig. 2).

At an early date Boston began widening its old streets and cutting through new ones to facilitate circulation. As a result of the fire of 1872 a large portion of the older city received wider streets, but with few changes in the general arrangement. In 1929, it was estimated that $7 000 000 had been spent on widening and straightening the streets to that time. At first, these improvements were successive, isolated projects. Several streets have been widened two or even three times, while other proposals have languished for many years before being acted upon. More recently temporary unofficial and official agencies have produced plans relating several improvements into a single plan for general relief. Usually, only a fraction of such projects was actually finished.

Work of Planning Board

Finally, following a compulsory State law sponsored by the Massachusetts Homestead Commission, a permanent official Planning Board was established in 1914, although without any powers other than those of advice. At first, ill-understood, it has gained ground gradually until to-day (1932), it is generally recognized as the central authority on planning matters for the city. At the outset the Board had to content itself perforce with promoting specific projects that appeared certain to be of value in any ultimate scheme; but, strengthened by its success in such undertakings as the Stuart Street and the Cambridge Street widening, it successfully launched a general zoning plan in 1924. Since then the Board has accumulated data and, with the aid of a special staff, it published in 1930 a comprehensive major street plan relating all street improvements to one another and to the city's ultimate needs (see Fig. 3).

Drainage Development

Other factors in Boston's plan became important long after its street plan was far advanced. The development of the system of land drainage,

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[4] *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2491.

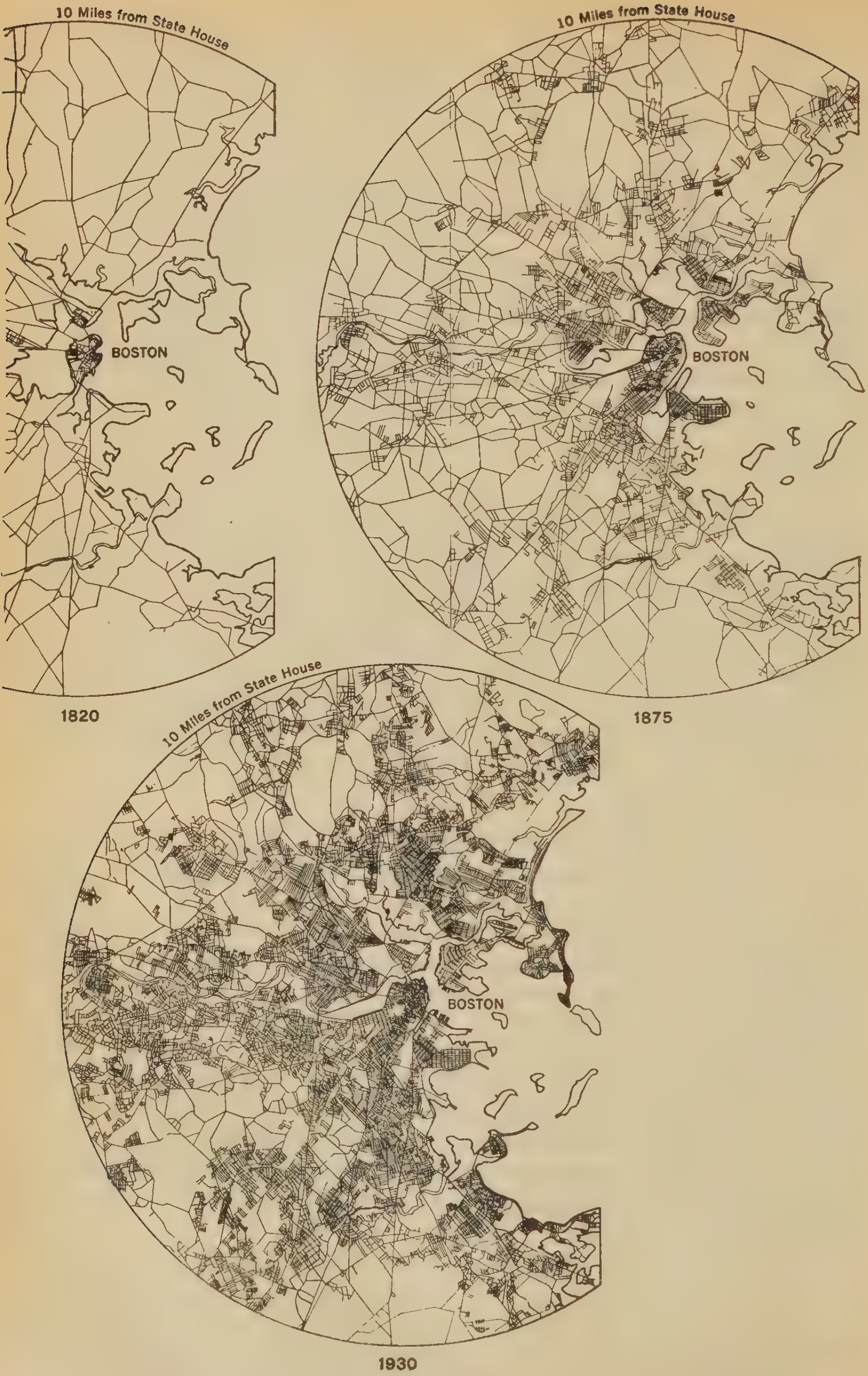
10 Miles from State House
BOSTON
1820
10 Miles from State House
BOSTON
1875
10 Miles from State House
BOSTON
1930

FIG. 2.—GROWTH OF BOSTON, MASS., AND VICINITY AS INDICATED BY EXTENSION AND DEVELOPMENT OF THE HIGHWAY SYSTEM.

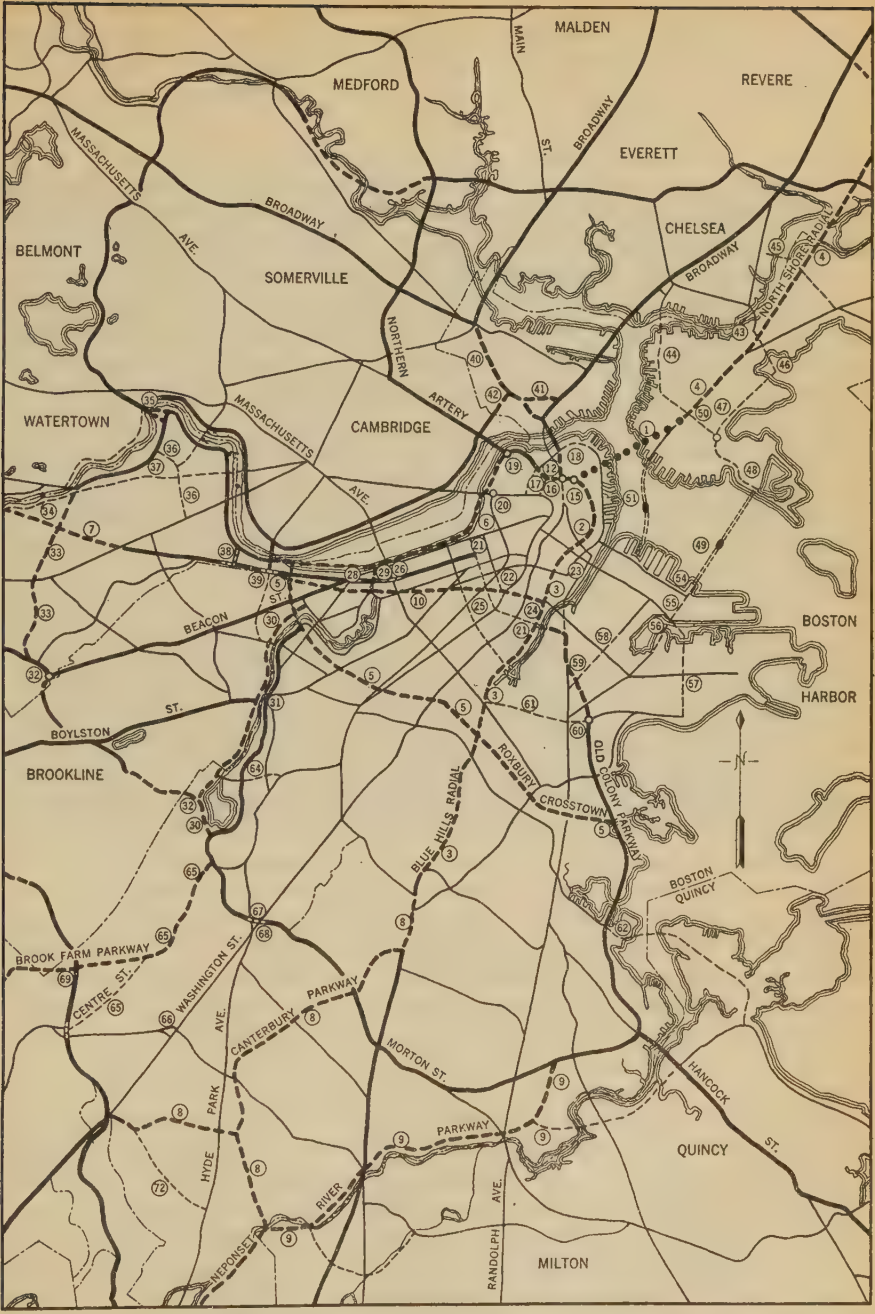
MALDEN
MAIN ST.
MEDFORD
REVERE
BROADWAY
EVERETT
MASSACHUSETTS AVE.
BROADWAY
CHELSEA
BROADWAY
NORTH SHORE RADIAL
BELMONT
SOMERVILLE
NORTHERN ARTERY
WATERTOWN
MASSACHUSETTS AVE.
CAMBRIDGE
ST.
BEACON
BOSTON
HARBOR
ST.
BOYLSTON
BROOKLINE
ROXBURY
CROSSTOWN
OLD COLONY PARKWAY
BLUE HILLS RADIAL
N
BOSTON
QUINCY
BROOK FARM PARKWAY
CENTRE ST.
WASHINGTON ST.
AVE.
CANTERBURY PARKWAY
MORTON ST.
HANCOCK ST.
PARK
HYDE
PARKWAY
QUINCY
RIVER
NEPONSET
AVE.
RANDOLPH
MILTON

FIG. 3.—THOROUGHFARE PLAN, BOSTON, MASS.

particularly as related to the larger watercourses, has exercised and still exercises a profound influence on the plan as a whole. At the outset the various estuaries proved to be serious barriers to highway communication, necessitating either long detours or ferries, and, later, bridges, which focused main roads at each end. For many years, while the mud-flats and marshes bordering these small streams were being gradually filled, the street system and the city as a whole turned their backs to the water, leaving it to the poorest, most casual development. Recently, all this has changed, however, first on a small scale, along Muddy Brook, now the beautiful Fens; then on a grand scale along the Charles River, and, to a somewhat less complete extent, the Mystic and Neponset Rivers and their larger tributaries. These stream basins have been reclaimed, lined with parkways, and have become among the district's proudest show-pieces. They form great "lungs" to the city's congested areas and provide ample space for great parkways for passenger traffic (see Fig. 4).

Plans approved in 1929 provided for still further development of the recreational service, as well as the facilities for vehicular movement along parts of the Charles River in particular. As the center of population of Greater Boston lies in the lower basin of the Charles, this water park effectively serves the entire metropolitan community.

Docks and Wharves

From the beginning Boston's docks and wharves gave a special character to the city as a port, but it is only in the fifty years since about 1880 that any general plan of port development has been followed. To-day, most of the water-front not devoted to recreation is developed or slated for development for wharfage for sea-going vessels.

Despite the dominance of New York, Boston persists in a slow but, on the whole, steady development as a port of importance. Although at first little attention was given to the need for relating port facilities to other elements of the city, in recent years much effort has been expended thus to render the port more efficient. Several of the largest filling projects have been for the purpose of reclaiming flats for docks, until considerably more acreage is now available than is in demand. One gain from this has been the location of Boston's newest transportation agency — the principal airport — advantageously close to the heart of the city in East Boston.

Railroads

When the railroads came to Boston these companies also found the purchase of tidal lands a cheap way to gain access. They are still active in filling areas for trackage. At first, each railroad was constructed as a separate undertaking, but corporate consolidations accompanied by physical unification have resulted in the two present passenger terminals: South Station and North Station. An expensive new building was erected for the North Station in 1928, thus probably crystallizing for thirty years or more the present none-too-satisfactory arrangement, particularly as regards suburban commuting travel.

At first, street cars simply fitted into the crooked street system, but this produced so much congestion that in 1898 the first American subway was built. Since that time all surface cars have been removed from the center of the city, and rapid transit lines have been constructed to tap all the inner suburban areas. The detrimental effect of elevated structures soon dictated the abandonment of that system in favor of tunnels and, in the latest instance, in order to economize expense, the open-cut system, utilizing existing railroad rights of way.

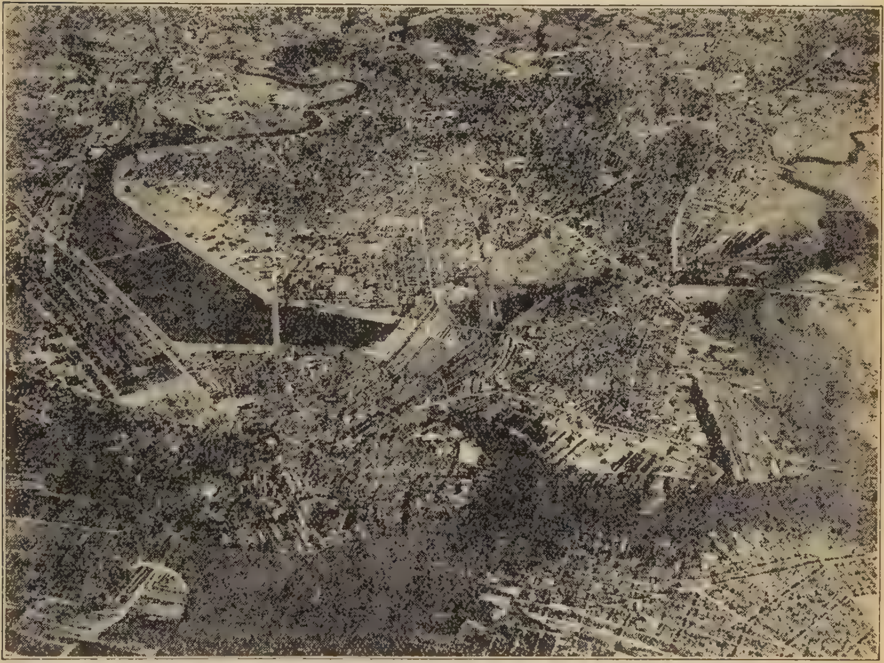

FIG. 4.—DOWN-TOWN BOSTON AND THE HARBOR, WITH CAMBRIDGE, MASS., BEYOND.

The development of sanitary sewerage[5] and water supply[6] has had comparatively little effect on the general plan of Boston.[5] Pipe lines, large and small, have followed streets rather than streets following the best alignment for pipes. To be sure, water supply reservoirs are important landscape features of the park system in several instances. Aqueducts have usually been buried, and the land often left in private hands, although one or two of the newest constitute unused opportunities for through walking routes extending many miles into the country.

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[5] *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2509.

[6] *Loc. cit.*, p. 2496.

PARK SYSTEM

As distinguished from circulatory services, whether of people or of goods, land uses in Boston, which are really the end product of a city plan, may be divided into two main classes—public and private. In so far as they are important in the city plan, public lands may be divided into lands for recreational service and lands for administrative service. Boston led the way in a connected series of parks, forming an actual system and, later, two interconnected systems—an inner city system and an outer metropolitan system. Most of these systems were planned before the days of automobiles when the present service to traffic rendered by the parkways was unforeseen. The parks themselves have continued to increase in usefulness for recreation, although the outlying ones are still in large degree what their generic name strictly means—"reservations" for the future use of a population that may either need them more or appreciate them more.

Boston was also a pioneer in supplying playgrounds, establishing the first municipal one in the country—the Charlesbank—in 1889. Others were added from time to time, and beginning about 1920, a vigorous attempt was made to place one within easy reach of every section of the city, even to the point of tearing down blocks of tenement houses to make the necessary space. Likewise, most of the available salt-water beaches, as well as a few of the inland ponds, have been developed for bathing, so that if present progress is continued steadily Boston will never be behind in providing recreation.

While the playgrounds are often close to schools, they are not, as a rule, a part of the school yard. Schools, therefore, have exercised less effect on the city plan as a whole, although certain large high schools, often constituting a group of buildings, vie with the colleges and other institutions sprinkled over the city in presenting effective civic architecture.

THE CAPITOL AND ITS SITE

Administrative buildings, in some instances, set a high standard for civic art. In Boston, the Capitol, or State House, stands pre-eminent on Beacon Hill (see Fig. 1). Threatening tall buildings have been kept away from it by condemning air rights above certain heights. The main axis of the building crosses the great central open space of Boston Common. Boston's charm is much enhanced by casual views of the dome of the State House as a dominant feature in the skyline across Charles River Basin, or of the building as a terminus of the street vista, as, for example, up Tremont Street from its intersection with Stuart Street. However, no other State or municipal buildings are grouped with the State House, and Boston has no civic center. Its City Hall, Court House, Post Office, and other important buildings are each tucked away on some narrow street or all too-small "square." Those few State functions not housed in the State House are located in such remote places as the Commonwealth Pier in South Boston and in scattered rented quarters.

ZONING

Boston early regulated the height of all buildings. A State law prohibits any building higher than 125 ft in any Massachusetts city, except Boston. In 1904, Boston really anticipated the zoning movement by dividing the city into two districts, with a lower height limit in the second. In 1923, the height limit in the less restricted district was raised to 155 ft, but with set-backs, if necessary, so that no part would exceed two and one-half times the distance to the farther side of the street.

Following a Constitutional Amendment and Enabling Act, Boston was zoned in 1924 under the writer's direction. Use districts were classified as Single Residence, General Residence, Local Business, General Business, Industrial, and Unrestricted; and bulk districts were classified according to height, as 35-ft, 40-ft, 65-ft, 80-ft, and 155-ft districts, each with appropriate requirements for open spaces. Subsequently, in 1928, the writer participated as Consultant in changing the maximum height limit to permit taller buildings within a pyramid-shaped prism starting 125 ft above the street, provided their volume did not exceed that previously permitted.

ADMINISTRATIVE PROBLEMS IN PLANNING

Two administrative factors have exercised a vital effect on the plan of Metropolitan Boston—its relation to the State and its separation into thirty-nine municipalities.[7] Due to the latter, progress in a unified development has been slow. Each municipality has worked out its own salvation, often with no regard to the others or to its own future as an integral part of a metropolis. Thus, radial highways are numerous, although narrow, while cross-town or circumferential connecting highways are either lacking or are imperfect, with numerous missing links. In the six years beginning with 1889, those services most imperatively demanding consolidation were so treated under metropolitan commissions. Twin sewage disposal systems were built to receive sewage from each municipality and carry it to sea. Most of the cities and towns join in a Metropolitan Water Supply System, the water being delivered to each municipality for local distribution. When a need for outlying parks, reservations, and beaches was appreciated another Metropolitan Commission was organized to secure these and, later, to connect them with parkways. Although these three services were put under a single head in 1920, they were each designed as a separate project and also without much relation to the other elements of the city plan. In 1929, the organization of a Metropolitan Transportation District was initiated.[8]

Owing to the preponderating importance of Boston and the District in the State, another factor in its development is still very much in evidence, namely, the State Government itself. The Legislature dictates many of the details of local activity, particularly whenever the project is of such size as to be outside the ordinary debt limit of the municipality. Thus, Boston's building code, zoning code, and many of its major street improvements, as well as practically all metropolitan projects, are controlled by special acts of the Legislature. After years of piecemeal work a Division of Metropolitan Planning was set up, which advises the Legislature on various large thoroughfare and transit projects for immediate relief both inside the City of Boston and elsewhere. This Division has prepared a comprehensive metropolitan thoroughfare and transit plan both as a guide for the Legislature and for individual communities. In 1929, another agency stepped into the field, the State Department of Public Works, with a plan for a radial highway northeast from the East Boston Tunnel, through East Boston, to existing highways in Revere, Mass.

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[7] *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2485.

[8] *Loc. cit.*, p. 2507.

City Planning Needs

One result of this multiplicity of planning agencies, with overlapping and sometimes conflicting fields of activity, has been to stultify local planning work except in Boston itself. Moreover, it has resulted in more than the usual confusion and expenditure of human energy in getting anything done. Again, as no one body is yet charged with the duty of preparing a comprehensive regional plan, only certain phases are covered at all. The planning of highways and transit receives close attention by the Division of Metropolitan Planning. The park system receives attention only in so far as it happens to relate to the highway system and to present parks and parkways. Metropolitan zoning of the use and bulk of buildings receives no attention although it is needed very much indeed.

Probably the most effective way to get ahead will be to charge the Division of Metropolitan Planning with the duty of preparing and keeping up to date a comprehensive Metropolitan Regional Plan and of constantly presenting this plan before the Legislature and to local planning agencies. Naturally, it would confine itself to problems affecting more than one municipality and would stimulate the local planning boards to develop and promote comprehensive city and town plans for their respective areas. The progress of Boston in developing according to a plan has been great. Its opportunity for further similar progress to meet current urban needs is even greater.

Acknowledgments

The writer wishes to record his appreciation to the City Planning Board of Boston, Mass., for courtesies in supplying copy for several of the illustrations in this paper.

DISCUSSION

ROBERT WHITTEN,[9] ESQ. (by letter).—In this excellent paper the author has referred briefly to the Thoroughfare Plan approved in 1930 by the Boston Planning Board. An outline of this plan is shown as Fig. 3 in Mr. Comey's paper. It is based primarily on a recognition of the need for a modernization of the present highway system by the development of a limited mileage of express roads and parkways of generous width, permitting a continuous flow of traffic. The widening of existing streets and the creation of traffic streets of the ordinary type have not been neglected, but these improvements, unless supplemented by those of the express-road type, cannot prevent a further increase in congestion, to say nothing of providing an effective relief for existing congestion.

Just as, about thirty-five years ago, street-surface congestion in Central Boston forced a start in the building of rapid subways to supplement the street surface car lines, so in this day the congestion produced by the automobile is certain to force the supplementing of the ordinary street system by a system of express roads.

A central feature of the Thoroughfare Plan is a great north-south road extending from the northerly city line bordering Revere, to the southerly city line at Readville, a distance of 13.7 miles. It will connect the State Highway System serving Revere, Lynn, Salem, Beverly, and other North Shore cities and towns with the State Highway System serving Stoughton, Taunton, Fall River, New Bedford, and neighboring towns on the south. It will also connect with a proposed new highway to Providence, R. I.

An important feature of the proposed Central Artery and its southerly extension by way of the Blue Hills Radial, will be an upper-level roadway or viaduct extending from the North Station at Nashua Street, Boston, to the Dover Street Bridge, a distance of about 2 miles. This six-lane viaduct will have a capacity of 60 000 vehicles per day, traveling at an average speed of 30 miles per hour. It will by-pass the chief centers of congestion and will attract to itself approximately 40% of the vehicles that are now (1933) clogging the surface streets of the central area.

The principle underlying the design of main trunk highways and parkways in the proposed Thoroughfare Plan is that they should provide, in so far as it is economically feasible, for a free and continuous movement of traffic. This requires that roadway levels be separated at the more important intersections. On the proposed Central Artery there will be a central viaduct over all cross-streets. On the proposed parkways, provision is made for over-passes or under-passes at all important intersections. For the Blue Hills Radial, the Roxbury Crosstown, and other major routes, a type of broad express road is recommended. For the locations selected this type of express road is believed to be the most economical and efficient means of providing the required traffic capacity, due consideration being given to the importance of speed, safety, and comfort of travel.

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[9] Planning Consultant, and Consultant, Boston (Mass.) Planning Board, New York, N. Y.

The problem presented is both that of providing relief for existing congestion and of providing traffic capacity for approximately 60% more vehicles than now use the city's streets. One way to meet this problem would be to widen a large number of existing streets from their present widths of 50 or 60 ft to 80 or 100 ft. Such wholesale widenings would be enormously expensive and would not remove the chief source of congestion and delay which results from crossings at grade. Under the conditions disclosed as the result of a careful analysis of the Boston traffic problem the construction of a few broad express roads seemed to be the only logical and practical solution.

A street system should not be designed to promote either centralization or decentralization. It should be designed to promote safety, comfort, and speed of movement, between all parts of the community. This will result in the best and most economical and orderly organization of the social and business life of the community. It will, in general, promote centralization where centralization is justified and decentralization where that is consistent with the best organization for the community as a whole.

A comprehensive thoroughfare plan is just as essential to the prosperity of the local business centers as it is to that of the main center. It is just as important to the local center that there should be adequate traffic-ways between it and the main center and between it and the other sub-centers as it is to the main center that it should be connected with all parts of the community.

It is sometimes said that it is useless to increase street capacities in central areas, as any additional capacity provided will be taxed immediately to the saturation point. This assumption may be valid in certain situations as applied to local business streets, but has no validity whatever as applied to any major traffic artery—to any street that is an essential part of a comprehensive thoroughfare plan. A serious slowing down of the traffic movement in any part of the main arterial system affects the entire community injuriously.

The automobile cannot serve the function now performed by rapid transit subways and elevated roads. These facilities must be relied upon for mass transportation along the most concentrated routes of passenger travel. Together with the surface cars and buses, they must continue to be the main reliance for the daily workward and homeward travel to and from Boston proper. The rapid transit system should be extended so as to reduce the rush-hour street traffic. There should be rapid transit routes and facilities to draw all trips that normally can be made more quickly, conveniently, and economically, in that way; and there should be express roads and parkways to accommodate all trips that normally can be made best by the use of the automobile.

The street system of the city should be adapted to the requirements of a motor age. The art of street design and construction has lagged far behind the art of vehicle design and construction. As a result, the citizens and business men of Metropolitan Boston are being denied the full advantage of one of the most marvelous developments of the age, the motor vehicle.

Industrial and commercial growth throughout Metropolitan Boston is dependent, among other things, on the development of a complete system of adequate and convenient traffic routes. The greatest potential advantage of a factory location in any great metropolitan center is due to the large home market within easy trucking distance. This is one of the chief factors in the continued growth of these great centers. Business growth and prosperity, in turn, are dependent on industrial growth. Convenient access between all parts of the Boston Metropolitan Area by truck and automobile is a prime essential for business and industrial prosperity.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1874

HIGH DAMS ON PERVIOUS GLACIAL DRIFT

By Edward M. Burd[1], M. Am. Soc. C. E.

With Discussion by Messrs. Charles W. Sherman, F. B. Marsh, M. M. O'Shaughnessy, H. de B. Parsons, Joel D. Justin, A. K. Pollock, William P. Creager, Adolph J. Ackerman, J. Albert Holmes, Irving B. Crosby, Harry H. Hatch, and Edward M. Burd.

Synopsis

Dealing in general with high dams on pervious glacial drift, the subject of this paper is based, to large extent, on Hardy Dam, on the Muskegon River, in Michigan. It is also based on the generalities of design and construction developed for such foundation by thirty years' experience with many similar structures of constantly increasing head and magnitude. This dam, which creates a 100-ft head, was placed in operation at full head on April 30, 1931. It is composed essentially of a porous sand embankment, 120 ft high, with a central flexible concrete core-wall. The core-wall is set on steel sheet-piling driven 60 ft into underlying, pervious, and heavily water-bearing sand and gravel glacial drift several hundred feet thick over bed-rock. Three penstocks, 14 ft in diameter, which are used both for power and spilled water, extend through the base of the deep section.

This type of construction, built on this kind of foundation material, is unusual. The difficulties involved have been solved mostly by experiment. The paper concerns matters of general design interest under similar conditions, such as foundation and embankment settlement, flexibility of structures, percolation, and spillway construction. Furthermore (and of equal or even greater importance), improvements and economics of design are suggested as necessary prerequisites to continued power development under the existing natural conditions and economic limitations.

Note.—Published in April, 1933, *Proceedings*.

[1] Civ. and Hydr. Engr., Consumers Power Co., Jackson, Mich.

INTRODUCTION

Dams should preferably be built on rock foundations—good rock foundations—as emphasized by past and present experience. Circumstances and conditions, however, have compelled the construction of many notable structures on various kinds of earth, and, notwithstanding the Biblical injunction, the horrible example therein described has been avoided in a commendably large percentage of such situations. Engineering in an area of glacial dumping grounds is largely a contest with this problem.

While many regions of the earth are devoid of accessible rock foundation, there is a belt across the eastern part of the border between the United States and Canada that has peculiarities of its own. Glacial drift is extremely deep, being as much as 1 500 ft in Michigan. While its deposition followed natural laws, the results are heterogeneous and seemingly at random to such an extent that any except the most general deductions beyond actual exploration are wholly unreliable. Most of the deposit was water-washed so thoroughly that the fines were removed and the residue became coarse and permeable. In this region natural drainage is geologically new and undeveloped, and ground-water lies close to the surface, usually permeates the entire deposit, and flows through it. Precipitation is fairly high, streams are swift, flow is unusually uniform, and water power markets are close at hand, all of which tends to foster dam construction on this unfavorable geologic structure.

Log-driving dams first, and, later, innumerable mill dams, all of moderate head, have been common throughout this region for fifty years. Many still survive, large numbers have been washed out for one reason or another, some were abandoned, and very few are still of economic benefit. None of them ever developed into structural problems in the field herein discussed, although a fair proportion have been re-developed into electric power properties on a moderate scale.

At the turn of the century, old mill dams on the Kalamazoo River, in Michigan, had been rebuilt into generating stations of the type in which a series of vertical-shaft turbines in an open timber penstock drove a single horizontal shaft with a generator at one end. Each turbine was harnessed through its large wooden-toothed bevel gear to a driven iron pinion on the main shaft. Over-all efficiency was not more than 50%; noise and vibration was considerably more than 200%; and the alignment of a long horizontal shaft over eight to twelve turbines, even if it was set straight (with penstock empty), on its log crib and earth foundations, became something fearful and wonderful with the penstock filled. Increasing power requirements of the industrial development of the Lower Peninsula of Michigan induced the design and construction of constantly increasing heads on the Kalamazoo, Grand, Muskegon, Au Sable, and Manistee Rivers, in the order named. In this evolution a center core-wall type of sand embankment on sand foundation was developed under the leadership of William G. Fargo, M. Am. Soc. C. E., for heads up to 48 ft, which was the limit beyond which dam construction, under existing conditions, was then considered impractical. Since then the Company with which the writer is connected, has finished a number of additional projects, modifying the design considerably as study and experience indicated. A plant of 41-ft head has been built on quicksand, another of 71-ft standing head was completed on clay and sand on the Manistee River, in 1926, and the plant on the Muskegon River, at Hardy, Mich. (100-ft head) was put into operation early in 1931. The latter design has a set of three pressure penstocks, 14 ft in diameter, on the original river bed through the deepest part of the embankment, and the foundation is of sand and gravel typical of an outwash plain. It is heavily permeated with water and its depth (perhaps 500 ft to bed-rock) has not been determined.

Topographically, still higher heads are possible on sites yet undeveloped. Structurally, present practise must apparently be considerably modified to adapt the design to higher heads, which, however, appear feasible and which offer a most interesting problem. Economically, the advance of steam generation has reduced the hydro-electric field to that of supplying peak power. This requires large ponds, high heads, large machinery capacity, and points toward future use of back-pumping during slack periods to supply water power for the next peak. Of course, water-power plants were built on the cheapest sites first, leaving each successive site less attractive as an investment unless new and more economical ideas could be devised. This general problem of geology, topography, design, and finance faces not only Michigan, but Wisconsin, Northern New York, and parts of Maine and Canada. Undeveloped water power approximately equalling those developed awaits its feasible solution. The purpose of this paper is to give in essential detail the experience developed in Michigan to date, and to suggest perhaps some possible changes and improvements.

Settlement of Supporting Earth Foundations

The formation of river beds in Michigan usually consists of a winding channel eroded from a glacial outwash plain, frequently following, in a general way, the toe of a terminal moraine which probably established the drainage arrangement originally. These outwash plains consist mostly of stratified, water-washed sands and gravels in all degrees of fineness, normally alternating with layers of so-called "mudstone." This latter material consists of water-borne glacial silt and rock flour deposited during periods of glacial back-waters caused by changes that are now more or less obscure. The weight of over-burden and the passage of time have compressed the mudstone layers into dense, tough, impervious beds into which a pick point can be driven only a few inches. When excavated, even in stream beds, the material is merely damp, not wet. When placed in water it disintegrates very slowly, but when pulverized and made into mud its rate of settlement and consolidation is a matter of geological time; for practical purposes "mudstone" remains fluid indefinitely. A 40-lb water-jet playing squarely against an exposed but undisturbed face for a week cuts away only a few inches. This material is distinctly different from morainal till clay deposits or clay resulting from rock distintegration in place. Boulders are almost never found in it, although occasionally they are found in the sand and gravel layers, where apparently they have been rolled along a stream bed. Casual observation may often show the present stream bed to be covered with a great accumulation of stones from material that has been washed away.

The foregoing describes the sections of the larger rivers in the Lower Peninsula of Michigan that are suitable for power development. Their smaller branches and other smaller streams flow through and over all types of glacial deposits, which fill the rock-stratum basin of the Lower Peninsula to a maximum depth of 1 500 ft. Lakes Michigan and Huron are merely glacial gouges from this deposit. Even the river beds north of the Grand Rapids-Saginaw line are elevated many hundred feet above ledge by a heterogeneous glacial dump that has no predictable formation or uniformity from point to point. Frequently, it is coarse and porous and heavily permeated with water. Nearer the surface the action that formed the outwash plain has given some continuity to the various strata, but, nevertheless, the mudstone layers are often irregular and fragmentary, due to subsequent forward glacial shoves or stream erosion after their initial deposition.

The selection of dam sites on this formation is largely a matter of arranging them so as to fit available mudstone, in the best possible manner, for foundation purposes. Surface topography in a river valley may be unrelated to the mudstone strata. In many cases the rivers have cut through these strata where they are most needed, and a series of ponds may necessitate a plant where only a sand foundation is available. Foundation design and safe head are largely matters of experience and necessity, rather than theory. Safety requires building for the worst existing condition as determined by borings, surface study, and previous experience.

Settlement under load on this formation fortunately proves less serious than might be expected, probably due mostly to previous consolidation under greater load before the stream eroded the valley, and partly to the water-settled nature of the material in outwash plains. The latter is an important factor. Material in the outwash plains is far different in behavior from that of till clays and unwashed morainal deposits, which have a much greater and uncertain range of settlement. Although test piles and test loading are used in each design, for well known reasons their indications are frequently inconsistent with actual construction adjustment, which constitutes the data herein presented.

Foundation settlement records on a number of dams in the region during the twenty years (1913 to 1932, inclusive) are typical of data shown in Figs. 1, 2, and 3, and in Table 1. These three illustrations and the table constitute the record to the end of 1932, of a 100-ft head partly on a thin stratum of "mudstone" supported on pervious water-bearing sand and gravel, probably 400 or 500 ft thick to bed-rock. The loading shown on Fig. 1 is the most severe yet imposed on this formation. Every detail was studied to avoid a large difference in load intensity at any point. Uniform variation of uplift as indicated on Fig. 1 has been determined experimentally by open-well pipes across the section of several lower but similar embankments. Borings were taken by "dry-sampling" pipes driven beyond the bottom end of 3-in. casings which were sunk by well-driving methods. The most pronounced difference

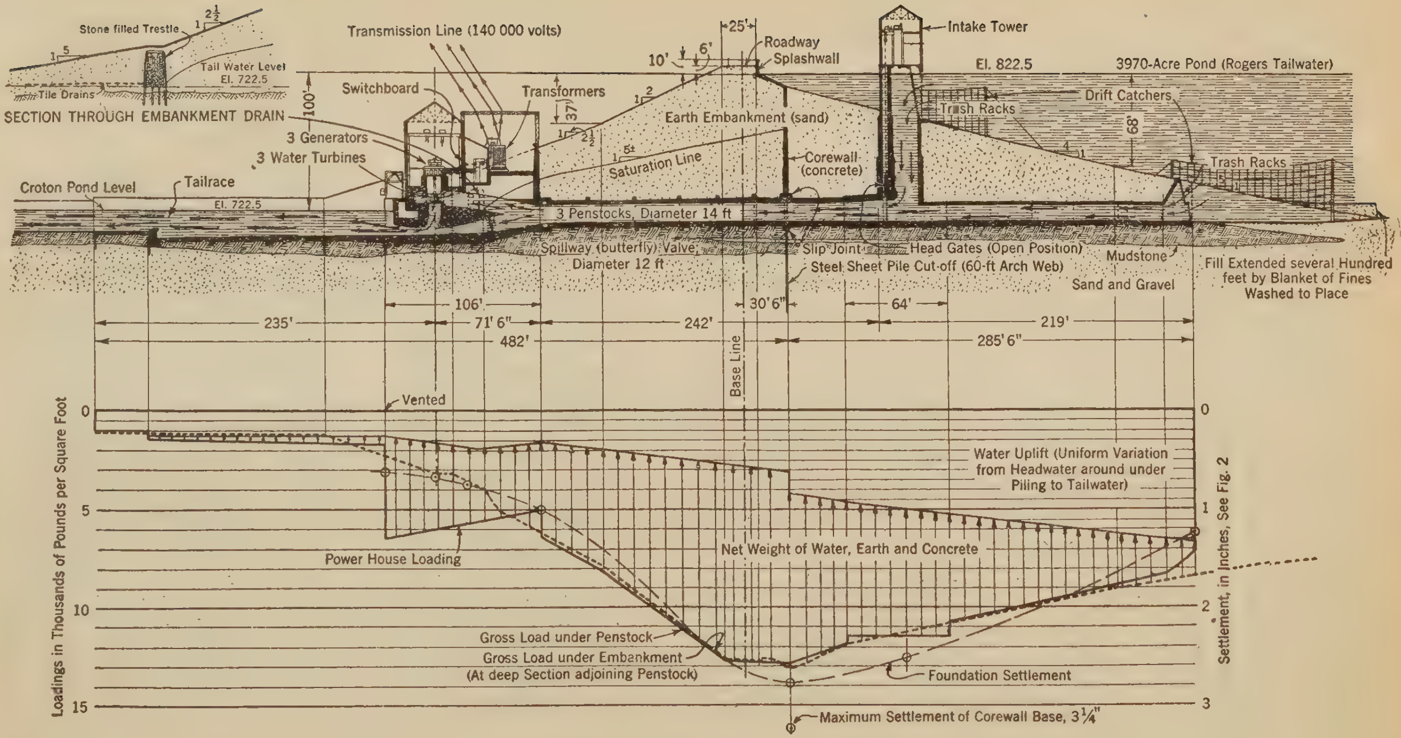
Stone filled Trestle
1
5
1
2½
Tail Water Level
El. 722.5
Tile Drains
SECTION THROUGH EMBANKMENT DRAIN
Transmission Line (140 000 volts)
Switchboard
100'
3 Generators
3 Water Turbines
Croton Pond Level
Tailrace
El. 722.5
Transformers
37'
1
2½
10'
6'
25'
Roadway
Splashwall
1
2
Earth Embankment (sand)
5±
1
Saturation Line
Corewall (concrete)
3 Penstocks, Diameter 14 ft
Intake Tower
El. 822.5
3970-Acre Pond (Rogers Tailwater)
Drift Catchers
Trash Racks
68'
4
1
Trash Racks
Spillway (butterfly) Valve,
Diameter 12 ft
Slip Joint
Head Gates (Open Position)
Steel Sheet Pile Cut-off (60-ft Arch Web)
Mudstone
Sand and Gravel
Fill Extended several Hundred feet by Blanket of Fines Washed to Place
106'
235'
71' 6"
242'
482'
Base Line
30'6"
64'
219'
285' 6"
Vented
Water Uplift (Uniform Variation from Headwater around under Piling to Tailwater)
Power House Loading
Net Weight of Water, Earth and Concrete
Gross Load under Penstock
Gross Load under Embankment (At deep Section adjoining Penstock)
Foundation Settlement
Maximum Settlement of Corewall Base, 3¼"
Loadings in Thousands of Pounds per Square Foot
0
5
10
15
Settlement, in Inches, See Fig. 2
0
1
2
3

FIG. 1.—CROSS-SECTION AND BASE LOADING OF HARDY DAM

is at the down-stream edge of the power-house block, where the expected differential settlement of about $\frac{3}{8}$ in. occurred. The maximum of about 3 in. under the core-wall was only one-half that expected, as explained subsequently under the heading "Design of Foundation Structures."

Settlements in Table 1 are total cumulative values, in feet, beginning at the date marked by an asterisk (*), for the years 1930, 1931, and 1932. Beginning dates are not shown for nine items in the table because of lack of room. To complete the record, these are: Items Nos. 21 and 22, March 25, 1930; Items Nos. 26, 27, 28, and 40, August 15, 1930; Item No. 34, August 30, 1930; Item No. 35, September 2, 1930; and Item No. 33, September 17, 1930.

It will be noted that, by averaging the final observations for Items Nos. 5 to 11, the average total settlement of the foundation was 0.28 ft. Levels taken at the penstock (Table 1(*b*)), on December 30, 1931, showed that there had been no change since August 1 of that year. Total settlements on the power house, as computed from Table 1(*c*), were as follows: Up stream, 0.085 ft; center line of power-house block, 0.065 ft; center line of units, 0.055 ft; and down stream, 0.050 ft. Likewise, the embankment (Table 1(*d*)) settled as follows: Items Nos. 46 to 49 (average), 0.160 ft; Items Nos. 50 to 52, 0.105 ft; and Items Nos. 53 to 57, 0.060 ft.

The foundations of the intake, penstocks, and power house were arranged on the only available mudstone, a bed about 20 ft thick, which "pinched out" both up stream and in the river channel, being the remains of a 40-ft stratum through which the river had cut. Such an arrangement is largely a matter of construction convenience. As far as load-bearing is concerned it serves merely as a pad or cushion upon the underlying sand, to smooth out local variations. Its effect in this regard is represented by about $\frac{1}{2}$-in. difference between $3\frac{5}{16}$-in. settlement of the core-wall foundation on sand and gravel and $2\frac{3}{4}$-in. settlement of the penstock foundation on this mudstone for practically the same loading. Total settlement to date (1933) is more nearly in proportion to gross loading intensity than net, which may indicate that water uplift is less than the uniformly varying pressure shown. This is probably true for the foundation on mudstone, but it is a condition unlikely to occur in porous sand.

Details of these settlement records are shown on Fig. 2, and in Table 1, which give the pertinent and typical points selected from a large number of field data. The trend of this record shows that settlement takes place in proportion to load. The embankment was all "roughed in" by November 30, 1930. After that date, and while the pond was filling, additional settlement of the core-wall base was only about $\frac{1}{4}$ in. More movement than this was expected while the pond was filling. Apparently, the uplift diminished the added water load considerably. The power house was completed by the end of 1930, as far as loading was concerned, the machinery and superstructure being finished early in 1931. It did not settle in direct proportion to its load, apparently being affected and controlled by general embankment settlement around it. (See Fig. 3 and Table 1(*c*).) No doubt the settlement of the embankment threw a heavy additional load on the up-stream side and may have reversed its base loading, which was desirable, but not assured by the

TABLE 1.—SETTLEMENT RECORD; FOUNDATION, HARDY DAM, MICHIGAN (CUMULATIVE SETTLEMENT OBSERVATIONS, IN THOUSANDTHS OF A FOOT)

Item No.	Year	1930																1931						1932
	Month**	July	August		September			October			November				December			January		Feb.	April	July	Aug.	December
	Day	5	6	30	4	10	24	3	17	30	7	14	21	28	5	12	29	13	31	27	1	7††	1††	29
	(a) CORE-WALL (SEE FIG. 2)																							
	Stations:																							
1..	19+30											*	0	0	0	0	0	5	....	....	....	....	....	Precise levels over thirty reference points show net average change from April 1, 1931, of 0.0035 ft, upward instead of downward, probably merely instrumental errors and no change indicated
2..	20+0							*	....	60	60	60	60	60	60	60	60	60	65	65	75	....	....	
3..	21+0										*	0	0	0	0	....	0	0	10	10	10	....	....	
4..	22+0										*	20	40	40	....	40	40	40	50	55	55	....	....	
5..	23+0					*	30	....	....	....	110	150	220	220	220	230	235	235	240	255	255	....	....	
6..	24+0				*	0	10	....	110	170	220	220	260	260	260	....	275	....	280	295	295	....	....	
7..	25+0			*	20	40	....	....	190	....	290	300	310	310	310	310	310	320	330	345	345	....	....	
8..	26+0			*	10	20	....	....	190	....	240	260	260	....	....	260	270	....	....	....	....	....	....	
9..	27+0				*	20	80	....	160	....	240	250	250	....	250	250	250	255	275	275	275	....	....	
10..	28+0				*	40	....	....	....	....	210	250	250	250	....	250	250	....	250	250	250	....	....	
11..	29+40	*	55	....	....	....	....	175	175	175	225	225	225	....	....	....	225	225	....	....	245	....	....	
12..	30+50	*	35	....	....	....	....	155	155	155	175	175	175	....	175	175	180	180	200	200	190	....	....	
13..	31+50	*	110	....	....	....	....	230	230	230	230	230	230	....	....	230	230	230	255	255	....	....	....	
14..	32+50	*	100	....	160	....	....	....	....	180	195	195	195	195	....	195	195	195	215	215	....	....	....	
15..	33+40	*	80	....	135	....	....	....	....	135	135	135	145	145	....	145	145	145	145	145	155	....	....	
16..	34+50									*	10	20	30	30	30	30	30	30	40	55	35	....	....	
17..	35+50									*	10	20	20	....	20	20	20	20	40	50	45	....	....	
18..	36+70									*	20	20	20	20	20	20	....	20	45	45	45	....	....	
19..	38+10									*	35	35	45	45	45	50	....	50	60	60	....	....	....	
	(b) PENSTOCK (SEE FIGS. 1 AND 2)																							
20..	Station: 24+71.8				*	....	....	....	110	....	190	210	220	220	....	220	220	....	....	....	225	....	....	Levels of December 31, 1931, and December 29, 1932, show no change at Bench-Mark 42; others inaccessible
21..	Bench-Mark: 42	‡	....	....	....	....	....	....	....	....	....	140	160	170	170	170	170	195	195	195	195	200	205	
22..	Up-stream end	‡	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	105	

* Beginning of test. ‡ For dates beginning, see text. ** Embankment construction period, deep (river) section, June 1 to November 30, 1930; pond filling, December 1, 1930, to April 30, 1931; power-house (substructure and superstructure), July 1 to November 30, 1930. †† No core-wall levels taken.

TABLE 1.—*Continued*

(*c*) POWER HOUSE (SEE FIG. 3)

Item No.	Year	1930														1931									1932
	Month**	September		October					November				December			January		Feb.	April	May	June	July	August	Dec.	December
	Day	20	25	4	14	23	27	31	7	14	21	28	5	13	31	15	30	26	1	1	1	7	1	21	29
	Bench-Marks:																								
23..	11								*	5	5	15	20	20	20	20	20	25	25	10	25	30	35	32	Bench-marks 11, 12, 13, 4, 5, 6, 7, 10, and two turning points average 0.003 ft‡ higher; probably instrumental error; no change indicated.
24..	12								*	0	0	15	20	20	20	20	20	30	25	25	30	30	25	35	
25..	13								*	0	5	15	15	15	15	15	15	30	25	25	20	25	25	29	
26..	14	‡	....	....	....	....	....	50	60	60	60	65	‖												
27..	15	‡	....	....	....	....	....	60	70	70	70	75	‖												
28..	16	‡	....	....	....	....	....	50	70	70	70	70	‖												
29..	4								*	5	5	....	5	5	10	10	10	15	20	20	20	20	20	22	
30..	5								*	4	5	1	15	2	20	20	20	25	.....	20	15	15	15	13	
31..	6								*	0	5	5	10	15	15	15	15	15	5	15	5	15	10	16	
32..	8								*	0	5	−5†	20	20	20	20	20	25	15	15	15	20	10	......	
33..	17	‡	30	....	....	....	....	60	55	55	55	§													
34..	1	‡	....	0	....	20	20	....	40	40	40	40	50	50	§										
35..	2	‡	....	30	30	30	....	....	40	40	40	50	....	50	....	....	60	.....	55	§					
36..	3	*	....	15	....	30	....	....	45	....	....	....	....	....	55	§									
37..	7								*	0	0	0	5	5	10	10	10	10	15	15	15	10	10	12	
38..	9								*	0	0	5	....	20	20	20	....	....	0	0	....	....	....	......	
39..	10							....	*	5	5	5	5	0	−5†	−5†	5	10	....	10	15	5	5	14	
40..	18	‡	....	....	....	....	....	10	30	30	40	45	§												

(*d*) EMBANKMENT (SEE FIG. 2)

Item No.	Year	1930								1931				
	Month**	November					December			January		Feb-	April	May
	Day	2	7	14	21	28	5	15	30	15	31	27	1	1
	Bench-Marks:													
41..	2	*	0	0	15	15	15	20	45	45	45	50¶		
42..	3						*	0	0	0	5	25¶		
43..	4						*	0	0	0	0	25¶		
44..	5						*	0	30	40	40	60¶		
45..	....											.....		
46..	16	*	0	0	30	55	85	110	140	150	160	160	150	195
47..	17	*	15	40	40	40	45	45	55	55	90	90	105	115
48..	19	*	0	20	20	60	75	75	75	85	150	150	155	165
49..	21						*	5	50	90	100	135	.....	155

Item No.	Year	1930			1931					1932
	Month**	December			January		Feb.	April	May	Dec.
	Day	5	15	30	15	31	27	1	1	29
	Bench-Marks:									
50...	23	*	10	35	40	60	60	95	95	Same as for "Core-Wall," no change. (For all Items Nos. 41–57)
51..	25	*	45	80	105	145	145	155	160	
52..	27	*	30	30	35	45	60	40	60	
53...	34	*	5	25	25	25	25	40	55	
54...	36	*	0	20	20	20	20	30	45	
55...	38	*	0	15	15	25	25	40	65	
56...	39	*	10	25	25	35	35	45	60	
57...	40	*	15	30	30	35	35	55	65	

* Beginning of test. † Bench-mark reported higher than it was initially (all readings in general being downward). ‡ For dates beginning, see text. § Covered by construction. ‖ Covered by fill; record transferred to Items Nos. 23, 24 and 25. ¶ Flooded out. **Embankment construction period, deep (river) section, June 1 to November 30, 1930; pond filling, December 1, 1930, to April 30, 1931; power-house (substructure and superstructure), July 1 to November 30, 1930.

design. As of the end of 1932 there has been no further foundation settlement. In general, the records show that settlement closely follows the loading and unlike more normal soils, including clays, settlement is not only less, but ceases much earlier. Fig. 4 shows these generalities which are typical of this kind of construction.

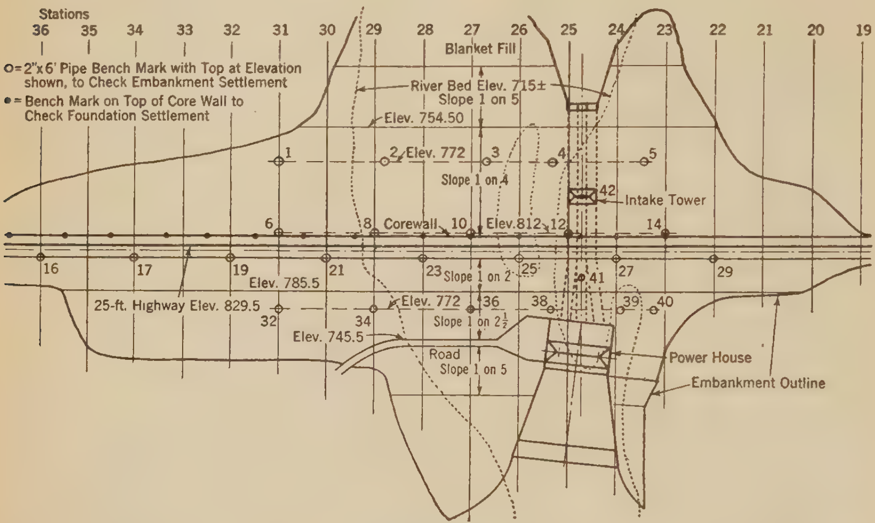
Stations
36 35 34 33 32 31 30 29 28 27 26 25 24 23 22 21 20 19
○ = 2"x 6' Pipe Bench Mark with Top at Elevation shown, to Check Embankment Settlement
● = Bench Mark on Top of Core Wall to Check Foundation Settlement
Blanket Fill
River Bed Elev. 715±
Slope 1 on 5
Elev. 754.50
2 Elev. 772
Slope 1 on 4
Intake Tower
Corewall
Elev. 812
1 2 3 4 5 6 8 10 12 14 42
16 17 19 21 23 25 27 29 41
Elev. 785.5
Slope 1 on 2
25-ft. Highway Elev. 829.5
32 34 36 38 39 40
Elev. 772
Slope 1 on 2½
Elev. 745.5
Road
Slope 1 on 5
Power House
Embankment Outline

FIG. 2.—SETTLEMENT RECORD OF HARDY DAM; PLAN OF STATIONS AND BENCH-MARKS (SEE TABLE 1)

The field method of obtaining these data was to establish reference points on the concrete base as soon as each "pour" was completed, tying them in carefully with several scattered permanent bench-marks well away from all construction. Reference points were replaced on each successive pour, being

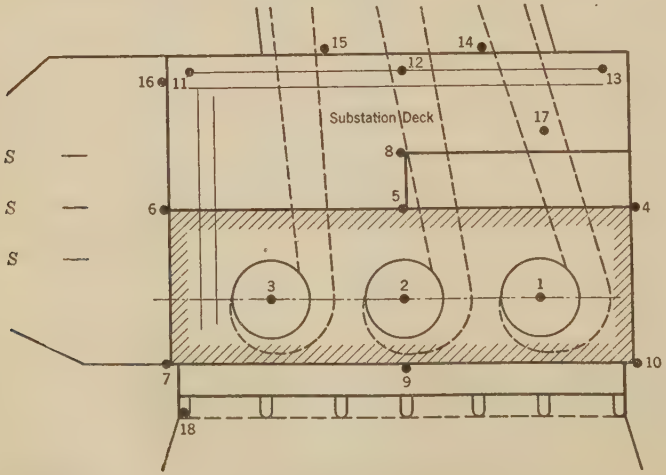
15
14
12
16
11
13
Substation Deck
17
8
S
S
S
5
6
4
3
2
1
7
9
10
18

FIG. 3.—LOCATION OF BENCH-MARKS IN POWER HOUSE (SEE TABLE 1(c))

carried in this way from the base to the crest of the core-wall, and assuming no appreciable change in the concrete itself after it was well set. The final points will continue to be observed until equilibrium has been established for a considerable time.

That settlement is proportional to load, and proceeds as loading is applied, and then ceases, even under these conditions, has been proved at several plants of lesser head, and under dissimilar conditions. In 1926, a plant of 71-ft head was completed on the Manistee River. It was founded upon a larger and more continuous mudstone stratum about 40 ft thick, beneath which is coarse sand and gravel carrying ground-water under strong artesian pressure, and to a depth not traversed by 100-ft borings. This power-house block is about 105 by 150 ft in plan by 110 ft high over all. It is set into the maximum embankment section, and it is of the open penstock type. The base pressure with full pond and penstocks is 9 500 lb per sq ft, and it is designed to give practically uniform gross pressure throughout the base. This block settled an even 2 in. along one diagonal; the opposite corners settled $1\frac{15}{16}$ in. and $2\frac{1}{16}$ in. Settlement ceased permanently after the pond was filled.

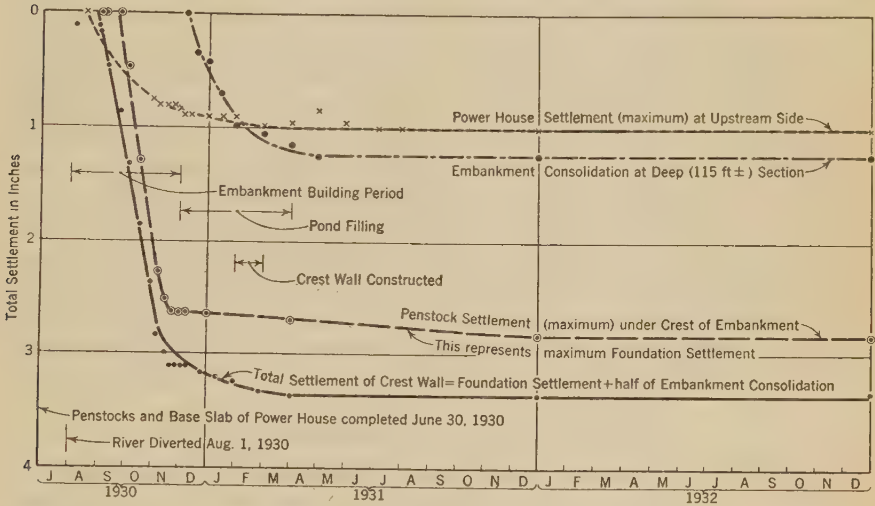

FIG. 4.—SETTLEMENT AT HARDY DAM

Another plant completed in 1923 was of necessity built on quicksand. The sub-surface formation consists first of 25 ft of ordinary river sand and gravel, beneath which is a similar thickness of heaving quicksand, and beneath this is again coarse and stable "water-sand," so-called. Any general loading of about 1 ton per sq ft on the top stratum will cause a slow settlement, displacing the quicksand, which is boiled to the surface by artesian pressure beneath it and escapes at numerous and ever-changing spring holes in the river bed down stream from the tail-race. Test piles showed adequate bearing in the water-sand below the quicksand. The design completely enclosed the base slab area (87 by 123 ft), with heavy arch-web steel piling, 55 to 60 ft long, which penetrated 5 ft into the "water-sand" stratum. A cut-off wall of similar piling extends each way from this box beneath the core-wall to the ends of the embankment. Inside the power-house box 1 266 long-leaf, yellow pine piles, 70 ft long, were jetted down and then seated about 15 ft into the lower "water-sand" stratum to a 20-ton bearing, by the *Engineering News* formula, $R = \frac{2\,wh}{s + 0.1}$, using a double-acting steam hammer, in which, $wh = 4\,150$ ft-lb. These piles are spaced on 2 ft 8-in. centers, with minor variations, to take the total base-slab load which is 8 500 lb per sq ft up stream, and 8 000 lb per sq ft down stream. The top of the steel piling is cast into the base-slab concrete, 2 ft in from the edge. This construction, although quite different from that on good sand or mudstone, as before described, settled almost exactly 2 in. while the load was being applied; and no settlement occurred afterward. These and similar records have established a rule-of-thumb ratio of 1-in. settlement for each 5 000 lb per sq ft of gross loading for any good sand, gravel, or mudstone. How far this rule could be extended and whether sand-box loadings for higher intensities would give comparable results are as yet uncertain. Flexibility of design details at Hardy Dam were arranged for many times the actual settlement experienced. As far as load bearing and consequent settlement are concerned, it seems safe and practical to increase the height of future structures still more.

DESIGN AND SETTLEMENT OF EMBANKMENTS

To build embankments on deep glacial drift is usually a larger and more difficult part of the entire development than is generally true under more usual conditions. The foundation not only adjusts itself to load, but is highly pervious both to pond seepage and to natural ground-water from adjacent hills. The latter is frequently the more difficult to handle, particularly where the embankment joins the natural banks. Valleys are wide even at dam sites, especially for the higher heads. Spillway requirements are a minimum, as floods in the common meaning of the word are unknown. This combination of circumstances results in an embankment problem of relatively large magnitude, involving structural difficulties and costing from 25 to 40% of the entire outlay.

Embankment materials are restricted economically to sands and gravels typical of outwash plains, which are unlimited in quantity, and of good quality, except that they are coarse and pervious. This class of earth is always near at hand; easy to dig, transport, and place; but it is, of course, water-washed and, therefore, low in fines commonly used for cores. Fig. 5 shows the composite fineness curve of all borrow-pits for Hardy Dam, as well as analyses of such well-known earth dams as Alexander (Hawaii), Calaveras (California), Taylorville (Ohio), Miami Conservancy District (Ohio), Saluda (South Carolina), Blue Ridge (Georgia), Davis Bridge (Massachusetts), Bridgewater (North Carolina), Soft Maple (New York), and Don Martin

(Mexico).[2] The curves permit a comparison of available embankment material in outwash plains of the glaciated region with samples from borrow-pits and cores of important and representative dams, built by semi-hydraulic methods. The embankment of Soft Maple Dam, which is about as high as Hardy Dam and which is also in the glaciated area, shows the quality of hydraulic sluiced core that is obtainable from available glacial deposits. When samples were gathered and combined at Hardy Dam, a small quantity of mudstone was included to represent that unavoidably excavated at the pits and transported to the dam, although the train haul and sluicing process could not pulverize these lumps and make them effective. Hence, the curve for Hardy Dam in Fig. 5 indicates more fines than actually are effective, and the curve for the dune sand at Muskegon, Mich., is perhaps more nearly typical. Even if all the fines could be washed out of the excavated material (which is impractical and uneconomical) the quantity would be inadequate for cores of the thickness ordinarily built by hydraulic or semi-hydraulic processes.

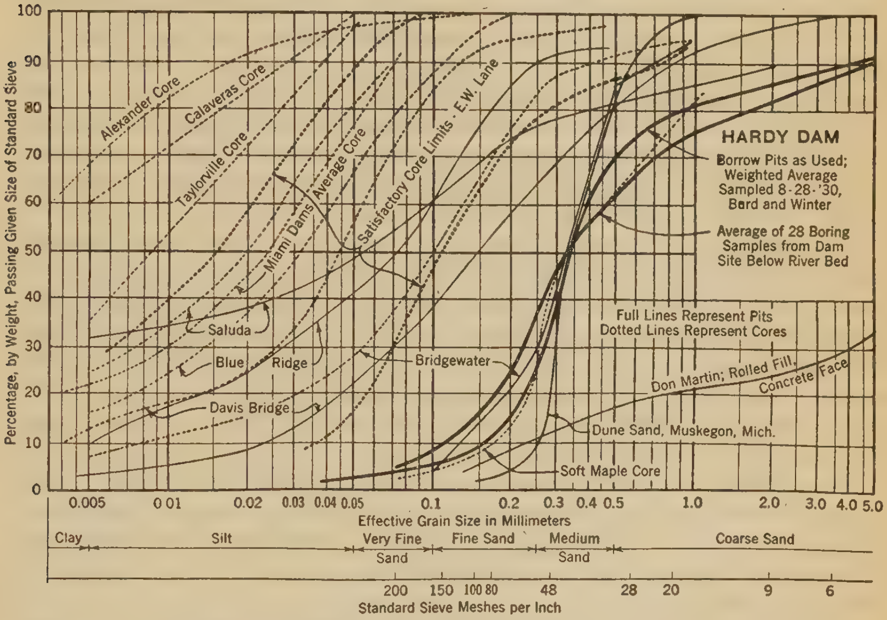

FIG. 5.—HARDY EMBANKMENT MATERIAL ANALYSIS

Only two embankments comparable to Hardy Dam have core or pit materials approaching its coarseness. Soft Maple Dam in Northern New York was built entirely by hydraulic-fill methods, but the similar available glacial material permitted a core scarcely any finer than the average of all the samples from the borrow-pits of Hardy Dam, as shown on Fig. 5. The resulting embankment seems to offer no advantage over the Hardy core-wall type, as to stability, economy, or tightness; while the rather high seepage loss approaches economic importance. The borrow-pit material at the Bridgewater Dam was only slightly finer than that at the Hardy Dam, and the resulting core lies well within the "satisfactory" zone of core limits. This would seem to indicate that similar results could have been obtained at Hardy Dam by semi-hydraulic methods. There is the difference, however, that the glacial outwash material has been leached of all alkali content that is present in normal soils, which induces a colloidal tightening not apparent in a physical fineness anaylsis. Furthermore, the usual semi-hydraulic methods used at Bridgewater Dam are not possible because of the cost, the first requirement being not only better but cheaper embankments. However, the possibility, recently developed, of alkali treatment for the up-stream embankment slope, is of interest and should be carefully investigated.

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[2] Hydro-Electric Handbook, by William P. Creager and Joel D. Justin, Members, Am. Soc. C. E., 1927, p. 249, Fig. 155; "Core Materials at the Germantown Dam," by C. H. Eiffert, M. Am. Soc. C. E., *Engineering News-Record*, December 18, 1930, pp. 954-958; "Materials in Existing Earth Dams," by E. W. Lane, M. Am. Soc. C. E., *Engineering News-Record*, December 18, 1930, pp. 961-965, and June 11, 1931, p. 962; and "Further Study of Earth Dams and Better Terminology Needed," by J. Albert Holmes, M. Am. Soc. C. E., *Engineering News-Record*, June 11, 1931, pp. 960-962.

To construct an impervious earth core dam under these typical glacial conditions evidently requires that additional fines be supplied either by pulverizing mudstone or by borrowing glacial till clays from morainal deposits which usually are much farther from sites than the sand borrow-pits. Either of these alternatives would complicate the operations and would add to the cost of a hydraulic-fill dam as usually built. Apparently, such alternatives never cost less than 60 cents per yd in place, in addition to rip-rap, toe drainage, and all other accompanying charges.

Proposals to dredge and pump material from floating equipment, giving access to fines in river beds and adjacent flats, have been advanced with extravagant claims. Probably this would be acceptable structurally, but when reduced to cost figures this method has never been able to compete with other methods.

The Michigan type of sand dam for structures of low head, with thin concrete core-wall, was originally devised to fit the peculiar local conditions most economically. A few have been built of moderate heads without core-walls, but with some serious failures and generally with unsatisfactory operating histories. The core-wall is constructed in sections and by vertical lifts as the fill advances around it. This wall is designed in sections about 20 ft square, well reinforced, with rods extending through the joints. This reinforcement is designed for flexure, and the joint is provided with sheet-metal water-stops. The wall is quite flexible and becomes adjusted to the fill on each side, which, in turn, ordinarily cannot be carried more than 5 ft higher on the high side. The embankment is constructed by filling both sides of the wall at one time, favoring either side as necessary in order to keep the wall aligned. The top also serves as a splash-wall; it eliminates the need of rip-rap, stops burrowing animals, and, to a limited extent, provides a self-supporting barrier which bridges any small washes and erosion until discovered and repaired.

The sand-fill is usually placed by the most economical means in each particular case. No special care is observed in sorting material as to size, but all fill is carefully washed to place with water. This requires moving the dry dumped material by high-pressure hose streams, and confining the flow inside of low shovel-built dikes that conform to the embankment section. Fill material has been transported from pit to section by gravity sluicing or "ground sluicing", wherever possible, this being the most economical of any method. Drag scrapers, mounted on tracks, as well as other forms of conveyors, are economical when topography and quantity justify their initial cost. Hauling material by train and dumping it from a trestle is the most flexible method although it is usually more expensive; but any of these methods, and usually some combination, has been used to place embankment material for a large number of developments at a total cost of not more than 30 cents per yd, and including the core-wall the cost does not exceed 35 cents. Earth dam construction has recently been discussed at great length in the technical journals, but almost without exception all reference to cost is omitted. Economy is as essential to good engineering as safety.

However, certain disadvantages of this type of core-wall design become more apparent as it is applied to increasingly higher structures. The part up stream from the core-wall merely supports it, adding little to general stability or percolation distance. At low pond stages, it is attacked by waves unless graveled and formed on a flat slope, not less than 1 on 4 as dictated by experience. The core-wall bisecting the embankment complicates and slows up the operation of placing the dirt and, as shown by test wells and piezometers, does not provide the drop in percolation gradient or saturation plane that might be expected. The ideal construction would evidently consist of some practical and economical method of grading the fill material from the finest up stream to the coarsest down stream (since an earth embankment is essentially a filter), protecting the up-stream slope against wave action, and providing it with a relatively impervious membrane having a seal at the bottom and ends of the embankment sufficient to secure the necessary percolation gradient. It appears certain that the same yardage of fill, and the same expenditure, with properly designed details along these lines, will secure a more stable and tight embankment. Engineering literature is replete with designs and adaptations that endeavor to accomplish this arrangement. Rock-fill dams, for locations where their use is economically determined, are now faced with concrete as standard practice.

Making haste slowly toward this end, because of settlement and adjustment problems under Michigan glacial conditions, the core-wall of Hardy Dam was moved 30 ft 6 in. up stream from the usual location and was "topped out" with slope paving to take wave action for any probable draw-down. The splash-wall, slope paving, and core-wall are all jointed and water-stopped. The relatively negligible settlement and improved tightness to date indicate that complete up-stream paving might well supersede the core-wall in future designs, both improving the structure and decreasing the investment. Of course, paving the face with concrete, for tightness of reservoirs and canals, is an accepted standard construction.

This type of earth embankment, paved for water-tightness, is not new; there are several examples of considerable size, and a number of smaller magnitude. As an example of the possible economies, consider the case of a face slab for the embankment of Hardy Dam, varying uniformly from a thickness of 12 in. at the top to 22 in. at the bottom and laid on a slope of 1 on $2\frac{1}{2}$ (which is conservatively flat). This slab would be designed with a 25-ft arch-web pile cut-off at the toe. A compilation of quantities for this design shows a construction saving of $150 000, an increase in percolation distance of 28%, and a symmetrical homogeneous embankment section unencumbered by piles and core-wall, for which construction convenience no credit allowance was made. If this type of embankment permitted a saving of 5 cents per yd in the placing of material (which is not unlikely), this would increase the saving by $75 000. Probably $200 000 would fairly represent the difference in cost.

About sixteen comparable examples of face-paved earth embankments are in satisfactory operation in the United States. The only essential difference is that they are all built upon supposedly non-yielding foundations; and the material composing the embankment is also supposedly more uniformly stable. Actually, this settlement record now proves that there should be nothing to fear from adopting this same idea for an embankment design such as that of Hardy Dam.

Settlement of the embankment of Hardy Dam was checked at a large number of points referenced by 2-in. pipes, 6 ft long, driven flush as soon as the fill was "topped out." A small but representative number of these points are represented by data in Table 1, as already discussed. Items Nos. 41 to 44, inclusive, Items Nos. 46 to 49, inclusive, and Items Nos. 53 to 57, inclusive, of Table 1, are all on fills of about the same depth, averaging 55 ft. Lack of exact uniformity is due more to variations in the care taken in washing the fill to place with water rather than to variations in the depth of fill. The area adjacent to the power house and directly over the penstocks, represented by the last five points (Items Nos. 53 to 57, Table 1), was given more care, the results showing in the fact that there was only a $\frac{3}{4}$-in. settlement, and that was quite uniform. Items Nos. 50, 51, and 52 are on the crest at a maximum depth of 125 ft, with an average settlement of $1\frac{1}{4}$ in. and a maximum settlement of $1\frac{3}{4}$ in., there having been no measurable increase from the last records shown—May 1, 1931, to the end of 1932. These results are representative not only of experience at Hardy Dam, but on many previous embankments composed of material from outwash plains of Michigan, when properly settled with water. These conditions insure a structure that is much more stable and free from excessive and continued settlement and sloughing than is usually experienced with well-built hydraulic or semi-hydraulic fill dams, of material ordinarily available.[3]

Percolation and drainage, however, are real problems in heavily glaciated regions. Except for its impervious element, every embankment composed of the available sand material is highly pervious, essentially a filter, as before mentioned. The seepage in this part can be predicted and controlled with much greater certainty than the seepage beneath and around the dam. In such regions the river bed is usually the discharge area for large volumes of natural ground-water that makes its way from adjacent higher levels through the deep and porous strata. Many of these streams are fed as much by this unseen flow as by visible tributaries. When a pond is raised, the entire water-table is likewise affected, and the water finds its easiest path of escape around the ends and along the down-stream side of the embankment. Fortunately, this usual soil formation can sustain a relatively high velocity of outflow without erosion and the springy areas of the natural surface, ordinarily, can be cared for permanently by applying gravel blankets and leading the outflow away in open ditches or tile drains.

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[3] See, for example, *Engineering News-Record*, November 14, 1929, pp. 769–772, and July 30, 1931, pp. 1047-1048; see, also, Creager and Justin's Hydro-Electric Handbook, Article 122, p. 254, and the accompanying bibliography.

The embankment itself should be under-drained to prevent percolation from sloughing away the lower part of the down-stream face and to increase general stability. Usually, this is done by laying bell-and-spigot vitrified sewer tile, with its non-cemented joints packed with marsh hay and protected by heavy paper or felt. The drain is laid in a V-shaped form similar to a pig trough, to preserve its alignment, and is back-filled with gravel. A grid of this construction, laid over about 25% of the total base thickness (or half way in to the normal core-wall location), has been uniformly successful in conducting away all seepage both through and under the dam. It is laid slightly above tail-water elevation so that it will wash itself clean, and preferably on a 1% grade, a sharp gradient being considered as undesirable as one that is too flat. At Hardy Dam a more generous down-stream trench drain was provided by filling the lower part of a dirt trestle, properly lagged with poles, with cobble-stone, out of which 12-in. tile headers projected at 100-ft spacing. This is indicated on Fig. 1. All drain outlets are preferably placed above tail-water, and are arranged separately and visibly for observation and measurement.

The seepage is highly variable as would be expected with the random and heterogeneous arrangement of glacial drift. Although the movement of water through soil has been determined with reasonable precision for any known condition, the haphazard occurrence of a glacial deposit which has been worked and reworked any number of times makes it practically impossible to apply the theory with any dependability. Providing for the maximum, based upon experience, seems to be the only certain method; and providing for it by laying the drainage system before the embankment fill is made—rather than fighting into existing seepage to lay drains—has also been proved, by bitter experience, to be not only safer but far cheaper. The seepage can be controlled in a general way by varying the percolation distance, which is the shortest water route from head-water to point of escape. After safety requirements are met, this control is an economic problem, because the loss of water is frequently appreciable. Part of the core-wall was omitted from a dam of 29-ft head, built in 1916. The initial seepage of 14 sec-ft was about 2% of the stream flow. Natural closure, without silt (the stream is clear), has reduced it to 7 sec-ft in fifteen years. The plant completed in 1926 (71-ft head) had an embankment 4 000 ft long and an initial seepage of 6 sec-ft; in six years this has decreased about one-third. Initial seepage at the Hardy Dam was 6.7 sec-ft, of which 3.7 sec-ft was percolation from the pond and the remainder from ground-water. In six months, it was reduced about 1 sec-ft, which was slightly more than is normally expected, and a smaller initial quantity than was expected, probably due to the deep and carefully driven steel pile cut-off. Since waters in glacial areas are normally clear, not much help from silting is ever realized. The effect of core-wall and pile cut-off is pronounced, as proved by the first case cited, and similar experience elsewhere. An impervious element in and below the up-stream part of an embankment seems not only necessary for safety, but is economically justified. In one case in which it was omitted, the leakage was about 20 sec-ft, or more than 10% of the stream flow in dry weather. The embankment was as high as that of Hardy Dam, one-third as long, and was built by semi-hydraulic methods.

In most cases no close distinction can be made between percolation through the dam from the pond, and general ground-water seepage. In mid-summer and mid-winter, temperatures will give a fair clue, since ground-water in Northern Michigan varies within the narrow limits of 42° to 46° F. It is usually slightly harder (in some cases, considerably harder), and often there is a slight color difference.

The hydraulic gradient, or plane of saturation, through these sand embankments is relatively high in the structure, and is marked by a separation of dry and saturated material that is quite definitely defined. It varies over quite a range in any vertical plane parallel with the axis of the dam. If the fill material were truly graded and of known size this seepage surface would presumably be regular, uniform, and determinate by analytical methods. Any such gradation of the wide range of random occurrence in economically available borrow-pits has seemed impossible to date, although, admittedly, it would be desirable theoretically. Any simple means to this end would greatly advance the art under these local conditions. Consequently, the saturation line as shown on Fig. 1 merely represents an average of many experiments with Pitot tube and open test pipes in a number of embankments. Sometimes, the core-wall lowers the saturation plane disappointingly little; elsewhere, under seemingly identical conditions, it is quite effective. Experience has indicated a safe generalization for this form and composition of embankment, namely, this plane remains below a 1 on 5 slope from head-water to tail-water; that is, the total travel of water around and under all cut-offs to the nearest point of escape on an embankment face or at tail-water should be at least five times the static head at the point in question. Embankments thus designed, without under-drainage, occasionally have a few damp or possibly wet spots at the toe.

The drains at the Hardy Dam reduce this percolation factor to 4.25, but the filter bed arrangement around the drain heads restrains the material effectively, as compared with a free surface. This factor of 5 is sometimes reduced safely; sometimes it may not be reduced. Many old dams in Michigan, built with little or no attention to design, have stood for years with a factor of 4, and sometimes as low as 3. The critical point usually occurs under the power house or spillway. Many of them have "blown out" from time to time, and usually upon investigation a spot in which the percolation factor is weak is shown by the evidence. A few dams without core-walls have been built with lower factors, usually with serious seepage intersecting the down-stream face. One 30-ft head so built blew out through the base of the embankment, with serious results. Any designer using a factor of less than 5 to free surface percolation would seem to be lessening the factor of safety, which should be protected in a dam, of all structures. Finer materials require higher factors. Several notable examples in the United States were designed with percolation factors of 8 or 10. In India, factors of 12 to 15 are common, all referring to relatively and uniformly pervious structures as distinguished from dams with dense and impervious puddled cores.

Surface protection on these sand embankments is a problem. The material is so sterile that even weeds will scarcely take root. If the slope is unprotected, wind carries away the fine sand until the remaining accumulation of coarser particles and small pebbles provides protective "rip-rap." The effect of rain is not as serious on this type of embankment as on clay surfaces, because the sand absorbs rain like blotting paper. On a long slope, however, rivulets begin to cut channels at the foot at times of heavy downpour, which erosion may then grow rapidly in size and seriousness, and must be cared for until a protective cover is grown. Berms spaced 20 to 30 ft vertically have been used, but they are expensive to build and require drainage maintenance. Slopes can be seeded by first spreading several inches of soil; but the cost is considerable during the first year for maintenance and care until a sod has taken firm hold. Seeding with a mixture of "red top" with alsike clover and some rye, seems to be the most successful method of establishing a cover. In the autumn winter wheat may be used. To plant "quack" grass (joint grass) is unlawful in Michigan, but there is no better seed for this purpose. However, relative costs of seeding and sodding at the Hardy Dam proved that under these adverse seeding and maintenance conditions, it was much more satisfactory from every point of view to place sod on the face. An abandoned meadow nearly a mile distant furnished the sod, which was cut 3 in. thick by machine, laid directly upon the sand, watered about two weeks, and apparently never stopped growing. Three acres were sodded at a total cost of slightly less than 2 cents per sq ft. The Michigan State Highway Department has established standards for sodding heavy cut-and-fill slopes, apparently for the same reason.

Design of Foundation Structures

Structures properly designed for construction on glacial deposits have proved stable and dependable practically without exception. Those few dams, general buildings, grain elevators, and similar heavy loads that have not secured adequate support on these formations constitute an extremely small fraction of the aggregate, and they indicate (as can usually be proved), that errors have been made in design due to a lack of understanding of the foundation material in question. Undue and disproportionate emphasis on the failures has established an unfair prejudice against it. The Biblical injunction against sand foundations applies to the stated lack of foresight that high water was to be expected. When they have had a clear conception of the fundamentals, adequate experience with glacial deposits, and a wholesome respect for peculiarities and conditions not always evident, engineers have achieved a high percentage of success in building on such foundations.

Settlement should be expected and must be provided for in the design. Limiting this broad subject to the still broad one of dam construction, the problem is usually further complicated by saturated foundation materials and horizontal pressures that induce a variable, and more difficult, distribution of loads. Design theory is largely empirical, being based on experience with the same or similar material elsewhere, and on the results of soil-loading tests, which likewise must be interpreted largely by experience. Relatively few precisely measured records have been kept and made available. In addition to the data herein given, a complete and excellent settlement record of Sherman Island Dam, on the Upper Hudson River in New York State, has been recorded[4] by H. de B. Parsons, M. Am. Soc. C. E. Numerous settlement records of embankments permit no separation of foundation settlement from consolidation of fill; but they appear to be largely or wholly the latter.

The extent and nature of settlement can be predicted with fair success, from boring samples, properly taken and preserved. Delays, failures, and great additional expense have been due to unreliable borings. For example, wash-borings, as usually taken, wash out the fines, disturb and re-arrange the material sampled, and mix it with an excess of water. The test-hole casing should be as large as essential data require and economy permits. An open-end sampling pipe should be used, as large as can be handled inside the casing. At the lower end it should have a short screwed section not more than 1 ft long, with a cutting-edge as sharp as can be maintained. After the casing has been cleaned, by the usual chopping and pumping operation, as near to its lower end as the material permits (but not beyond), and has been bailed out this dry sampling pipe should then be driven into the dry and undisturbed material well beyond the lower end of the casing. The pipe is then pulled and the sample plug is removed from the nipple point as carefully as possible. It is placed at once in a glass jar with a tight cover and properly identified. Casing is then driven a distance determined by the sampling interval, and the process repeated. Many jobs must be prospected with the aid of a well driller and his outfit. Without instruction and inspection, as herein outlined, his record is usually unreliable and sometimes dangerously misleading. On the other hand, after the well driller becomes familiar with this method, the additional labor required ordinarily increases the cost of prospecting less than 25 per cent.

After the foundation material has been so sampled to a depth equal to the head contemplated, a visual examination usually indicates the settlement conditions to be met in the design. More precise methods would be justified in more consistent and continuous deposits, but glacial drift is almost without exception so irregular in its characteristics that the worst, and not an average or general, condition must be met.

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[4] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), pp. 1257-1292.

Mudstone, gravel, or coarse sand alone have proved suitable for plants up to 100-ft heads, and are desirable, in the order named, for construction reasons. Ordinary stratified mixtures of these materials have consistently resulted in settlements of 1 in. per 5 000 lb of gross load uniformly distributed over large areas. At the Sherman Island Dam previously mentioned a uniform load on similar material, slightly less than 5 000 lb per sq ft, indicated a final settlement of 1.4 in. Mr. Parsons explains, however, that the reference points were on the top runway of the slab-and-buttress concrete structure, 80 ft above the foundation, and, hence, included any elastic adjustment within the structure itself. At both the Hardy Dam and the Sherman Island Dam the settlement records can be adjusted to a gentle easy curve proportional to loads within the range of use (as much as 13 000 lb per sq ft at the Hardy Dam).

Fine sand, grading down to quicksand, is treacherous of course. When it is saturated and when it has an excess of active ground-water, the ratio of pressure induced at right angles may increase to unity. Ordinarily (and preferably to be avoided), foundations on this material, and for moderate loading, can be built and prevented from settling if the bearing area and material can be confined to prevent lateral escape. Thus, as previously mentioned, at a dam completed in 1923, a 20-ft active quicksand stratum, beneath a 25-ft surface of ordinary sand and gravel, was confined by steel sheet-piling, and the enclosed area supports a uniform load of nearly 5 tons per sq ft, on round timber piling. Each foundation condition and design of this kind is a different problem not subject to generalization.

Soft clays and mud, silt, peat, and muck are frequently and generally found throughout the glaciated area. Moderate, quiescent loads can be floated on these materials, but settlement seldom follows a well-defined or dependable program, except that it is likely to continue indefinitely. Most of the foundation trouble in this region has been in connection with this class of material. Highway and railway fills sometimes settle for years, raising billows along each side, and this settlement continues until the unstable material is displaced. A common method of handling fills on such material is to place a part or the bulk of it and then blast the soft layers beneath. The shock of the blast and the superimposed load serve to squeeze out the soft material quickly and to provide a bed for the good material immediately. To construct any foundation on this class of material would be disastrous for a dam of any magnitude, not only as a matter of bearing, but also as regards sliding.

Substructures on sand, gravel, and mudstone should be founded as carefully as possible on undisturbed material, to avoid localized differential settlement. Thorough under-drainage, as previously described, is usually a necessary construction measure, despite the fact that, sometimes, this is not desirable from the standpoint of design. Percolation factors and bearing intensities must be adjusted accordingly. Substructures should be designed as floating units. Adjoining units must be detailed for relative motion considerably in excess of that expected. All changes of base pressure should be accomplished gradually. Base pressure at points adjoining earth fills should be approximately the same as the load exerted by the fill itself, to avoid a plane of rupture due to differential settlement, which is a common cause of failure. The danger of sliding is usually provided against easily, by providing cross-trenches on the bottom of the base pad. Percolation along transverse planes of contact between fill and concrete is blocked by a generous use of "fin" walls.

The foregoing requirements were applied to the Hardy Dam by designing the substructure of the power house and intake tower as completely self-contained, rigid boxes. Although placed in a number of different "pours," each construction "run" is so thoroughly tied into the entire structure, that the whole is virtually a boat, and by actual precise levels, this and other similar substructures actually tilt with each successive "pour." By careful arrangement of the order of "pours" it is possible approximately to hold an "even keel" so as to favor points of joining adjacent structures. The penstocks were designed and built in 30-ft sections, joints being detailed to permit settlement and adjustment by special provisions in both steel and concrete. Soft pine form lumber was left embedded around the top half of joints of concrete, so that it would compress with settlement. Steel liners were fabricated and assembled so that considerable angular adjustment could take place at each joint. The amplitude of possible movement was based on a possible settlement of 5 ft under the crest of the dam, although only 6 in. was the maximum expected, and about 3 in. was all that was actually experienced. A 6-in. upward camber was put in the penstock grade and unwatering the penstock affords a check that only half has been removed by settlement of about 3 in. as recorded. Precise levels the full length of each of the three penstocks show easy and almost identical curves as indicated by the average in Fig. 1. Concrete joints in the penstock were matched to prevent any tendency to offset; no such tendency has been indicated. The core-wall was notched by tongue and groove into the penstock shell to permit any relative movement (of which there has been $\frac{1}{2}$ in.) and still maintain water-tightness. Although the steel "liners" in the penstock were heavily "ribbed" with angle stiffeners and otherwise anchored into the concrete shells, no attempt was made to make them absolutely water-tight. In fact, the parts up stream from the core-wall were perforated with $\frac{1}{4}$-in. holes on about 8-ft centers to relieve any water pressure between "liner" and shells. While these precautions are somewhat unusual, and even though jointed, adjustable penstock sections are not general, no distress has been evident to date (1933), as distinguished from experience elsewhere with rigid, non-adjustable, and unvented designs.

To require that a power-house structure be designed to insure all pours being knit together into a rigid self-supporting whole, imposes a much more severe condition than if the foundation was rock. Loads must also be arranged so that the combined load and thrust will intersect the base near the center, in order to preserve plumb and level conditions as far as possible. The substructure of the power house at the Hardy Dam was determinate as to its loads and thrust, but the effect of the adjacent embankment loading was indeterminate. Its estimated value was actually exceeded, with the result that the structure tilted up stream $\frac{5}{8}$ in. in a width of 106 ft. This is less than is frequently experienced with power-house structures of the open penstock type, built entirely through the embankment.

This tilting during the construction of practically all substructures on Michigan rivers is of no disadvantage, except as it concerns the usual methods of erecting power-house machinery. Erectors normally proceed by plumb and level methods. In fact, at first any other program seems impossible to them. Turbine discharge rings are always set level; then speed-rings are set to them, or sometimes *vice versa*. Several months elapse before the substructure is complete around the penstock, scroll case, and pit liner, ready for the turbine assembly. By that time the final weight has largely been added and its distribution may be decidedly different from what it was when the discharge ring was set. Consequently, the erector then finds his contact surfaces neither plumb nor level, and he either throws up his hands in disgust, or insists on shimming everything back to plumb. He would not appreciate that still further loading, and consequent adjustment, would change this condition again, before the unit was erected, and would make future dismantling and assembling almost impossible. In time, the erector understands that all parts should be set square and concentric with the speed ring "as is," and that they should be properly match-marked and doweled for future assembly. With many units thus assembled, no operating disadvantages have ever been noticed for the slight amount that a vertical shaft is out of plumb.

Uniform distribution of base-slab loading, over wide draft-tube spans, is often best accomplished by a liberal use of heavy steel beams or open steel trusses embedded in concrete. Embedded steel trusses are used where necessary to secure the end-to-end stiffness of the entire structure. Fortunately, the use of steel in these structures whether reinforcement, structural, or plate, is free from the dangers of corrosive action caused by less pure waters and soils that are not water-washed as is this glacial drift. Experience of thirty years shows practically no rust on steel maintained 5 ft, or more, under water, even without paint or protection of any kind. A greasy scum, apparently of animal life origin, provides a protective coating. At certain seasons when pits are unwatered for inspection the turbine-case castings are literally covered by small white grubs. Structural plates, washers, and bolts, not protected by paint and removed after approximately twenty years, had mill scale intact under this coating.

Round timber piling is used as little as possible for substructure support, and then only when any possible water channels beneath its supported structure are effectively blocked by steel pile cut-off walls. Furthermore, its resistance to lateral thrust is so uncertain that all such thrust must be directed elsewhere. Round pile foundations, however, are widely used for general construction in the glacial drift areas, where material is too soft to support loads on spread footings, which are given preference when suitable. When timber piles are driven below ground-water level (and this would apply practically to the construction of all hydro-electric plants), there is no measurable deterioration, and there is no teredo organism. Logs are still being salvaged commercially from river beds, where they have lain since lumbering operations in the Seventies, and they are structurally as sound as new timber, although the sapwood sometimes is stained. Round piles usually have an ample factor of safety even when loaded to the capacity indicated by the *Engineering News* formula. Any types of sand and gravel afford high skin friction and, usually, driving is best accomplished by jetting, continuously, up to the point of seating. Even in quicksand, the pile is seized rigidly when jetting ceases, and, frequently, it cannot be started again without jetting.

Steel sheet-piles are used generously for cut-off purposes. In fact, without modern piles and pile-driving equipment, dam construction on these glacial deposits would be limited to plants of relatively low heads. Steel pile cut-off walls prove most effective in blocking under-flow, and the heights of structures now seem limited only by the depth of the cut-off that can be driven (at present limited by practical considerations to about 60 ft in sand and gravel). Driving is easier in clay and even in mudstone, but sand packs in the interlock between steel piles with a grip that practically stops driving at a penetration of 60 ft, even with a double-acting steam hammer striking an 11 ft-ton blow. No known method of jetting, and no form of interlock yet devised, will keep that interlock free beyond this limit. Jetting is an essential part of deep driving in these formations, not only to reduce skin friction and to free the interlock, but also to permit drawing an occasional boulder to one side. While not numerous below the accumulation on the river bed itself, there are enough boulders to cause some trouble and concern. Although some engineers have doubts as to the efficiency of steel pile cut-off walls for this reason, results show that an experienced driving foreman can tell when he hits a stone, can guard against "jumped" interlocks, and can detect them quickly enough if he is conscientious. Piles are sometimes pulled back as a test of driving results. In one case of extreme and unwarranted caution, all the piles (only 25 ft long) on a recent dam job in Southern Michigan were pulled, inspected, and re-driven, the cost in this instance being apparently of no concern. A moderate use of such methods, or even the threat of them, will guard against breaks in the cut-off wall. Mandatory open-trench cut-off construction, as on the Bingham Dam, in Maine, would make Michigan projects impossible economically.

Pressure grouting with cement, mud, asphalt, etc., has been tried, but with little success because the grout goes into the more porous openings of least resistance which, almost without exception, are filled with large quantities of ground-water under head. This water is usually flowing with considerable velocity toward some point of escape not far distant. The excavation of grouted areas shows merely a series of discontinuous and random masses that are of little use as a continuous cut-off. Up-stream earth blankets are good, but are scarcely dependable unless backed up by a steel cut-off wall. The combination is practically water-tight, however, because the interlock becomes packed with sand so completely as to plug even its slight clearance. Steel piles are not used for bearing, except in some relatively unimportant position. Under the core-wall and penstocks, a rough wood box form to exclude concrete is placed over the top to provide a slip joint and this form is left in place. The concrete footing of the core-wall secures a grip on each side of a pile, below the top, and keeps a tight joint if the core-wall settles over it. When steel piling is used at, or above, the water line, it is galvanized. When used for revetment walls to maintain river channels, canals, and raceways, the results of this system are good, and economical.

The Spillway

In Lower Michigan, insufficient spillway capacity, as far as known, has never caused the failure of dams of any magnitude or of dams built on the basis of engineering design. Numerous old mill dams have washed out from time to time, from various causes, and while, unfortunately, no distinction is made by the general public, the design of these structures is seldom based on engineering study.

Application of capacity requirements based on engineering study must be modified, of course, for such matters as ice, rubbish, frozen-in gates, etc. Heavy ice forms on power ponds in this latitude, but pond ice has never been known to run on the Muskegon, Manistee, or Au Sable Rivers. Break-up flood flows are insufficient to move it down river, and it melts in place. This would scarcely be true of smaller streams, with smaller ponds, even in the same vicinity. This pond cover effectively prevents the formation of frazil and slush ice. Lumbering rubbish is an aggravating nuisance on these rivers, but with boom control it has never interfered with the opening of spillway gates, although it has interfered with closing them. Free crest spillways would insure against these possibilities, but these have not been required. Gated control, which has decided advantages and economies, has been used almost entirely. Steel Taintor gates have been standardized in a size, 21 to 24 ft wide by 13 ft high, with a capacity of about 4 000 cu ft per sec per gate without head rise. First cost is so moderate, operation is so simple, and maintenance is so low that it is difficult to visualize the necessity for the complicated and expensive roller type which has been installed in many developments.

As far as size and capacity are concerned, Taintor gates have been built and operated with complete success in Northern latitudes, and in ice-laden rivers, in sizes up to 80 ft long and 30 ft high. Their weight is only a fraction of that of the other type, and the setting and hoisting details and equipment are far more simple. When properly designed and detailed, they are entirely determinate in stress analysis, decidedly economical in structural design, as tight as any other gate, free from damage due to spilling débris, and low in maintenance. Means have been developed to prevent their freezing in, and the same measures can be applied to them as easily as to other types. Recent advances in welding have further decreased their cost and improved their tightness. On one hundred spillway gates of this type, and on thirty penstock gates (usually 30 by 30 ft), within the writer's experience, no Taintor gate of this design has failed structurally, or has refused to operate when required. Under-sluices beneath power houses have been used in a few plants. They permit closer and easier pond control for small waste discharges without outdoor spillway operation.

Spillway design on glacial drift is somewhat more difficult than on stronger foundations. The structure must be sealed and provided wth a cut-off wall that is even more conservatively designed than for the embankment, since percolation distance under it is usually shorter than beneath an embankment. Generous use of steel piling and cut-off fins provides these requirements. By using a buttressed and inclined-slab type of concrete structure, foundation loading is held within bounds. For the higher heads gravity sections cannot be supported, even if they were otherwise desirable. Spilled water energy cannot be turned loose on foundation material, but must be dissipated by stilling pools or by baffles. Outflow velocities must ordinarily be held to a maximum of 4 ft per sec, and kept straight and uniform off the end of the apron. Tumble bays were built at earlier plants to absorb energy. They were designed before model testing was developed to its present high state, and many were only partly successful. Submerged weirs were used later to take the shock of discharge and to create standing waves. These were tested by means of models and were more satisfactory. More recently dentated sills (not necessarily of patented design) have been used for the same purpose and with most satisfactory results. A dentated sill only 46 ft from the free discharge end of a 6-ft spill tube, operating under a full head of 150 ft, destroys the energy so effectively that sand collects almost up to the level of the apron slab between the sills. Dentated sill designs, developed and tested by models, have greatly improved spillway conditions on soil foundations.

Further study and model testing are likely to develop some arrangement similar to that shown in Fig. 6. The construction trestles in this section are placed in echelon and are built to a height and spacing that will fit the topography and will permit progress from the ends toward the center, using cross-dikes. The pit-run sand for the embankment is delivered from the trestles and sluiced up stream on a slope of about 1 on 8, thus forming a blanket of fines over the base and grading the finest material up stream throughout the height of the embankment.

For complete segregation of the washed portions (practically attained by washing "to place"), the fineness gradation would be about as shown in Fig. 6. The interlocked steel sheet-piles at the up-stream toe would be of a length dependent upon the sub-surface formation. In general, they would be long enough to insure a percolation factor of 5 to drains. These piles are driven after the fill has reached the elevation of the top of the piles, and the top is cast into the paving slab as shown.

The up-stream slope is diked and vented during construction and the excess material over the neat grade is placed in a blanket to form a buttress and seal at the toe.

The concrete paving is laid in articulated tongue-and-groove slabs, well reinforced, and provided with crimped metal water-stops in all joints. The slab is well keyed into the slope to prevent sliding.

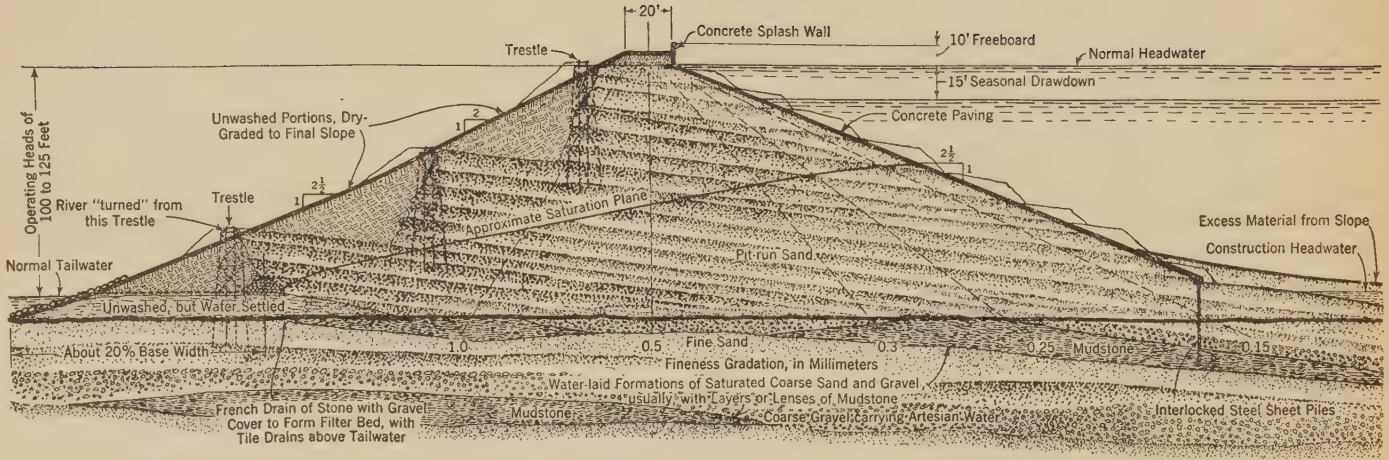
20'
Concrete Splash Wall
10' Freeboard
Normal Headwater
Trestle
15' Seasonal Drawdown
Concrete Paving
Unwashed Portions, Dry-Graded to Final Slope
2
1
2½
1
Operating Heads of 100 to 125 Feet
River "turned" from this Trestle
Trestle
2½
1
Approximate Saturation Plane
Excess Material from Slope
Construction Headwater
Normal Tailwater
Pit-run Sand
Unwashed, but Water Settled
About 20% Base Width
1.0
0.5
Fine Sand
0.3
0.25
Mudstone
0.15
Fineness Gradation, in Millimeters
Water-laid Formations of Saturated Coarse Sand and Gravel, usually with Layers or Lenses of Mudstone
French Drain of Stone with Gravel Cover to Form Filter Bed, with Tile Drains above Tailwater
Mudstone
Coarse Gravel carrying Artesian Water
Interlocked Steel Sheet Piles

FIG. 6.—IMPROVED SAND EMBANKMENT ON TYPICAL GLACIAL OUTWASH FOUNDATION

CONCLUSIONS

On the basis of the foregoing arguments the writer offers the following conclusions for discussion:

(1) Coarse sand, gravel, or glacial mudstone river-bottom formations support loads satisfactorily if such loading is graduated gently to a maximum of 13 000 lb per sq ft, with an average allowable settlement of 1 in. per 5 000 lb per sq ft of loading.

(2) The performance of this type of foundation indicates the probability that tests may develop still greater load-carrying ability without excessive settlement or other difficulties.

(3) Settlement is essentially proportional to load and takes place as the load is applied; it practically ceases thereafter.

(4) Sand embankments, composed of river valley and glacial outwash material, settle in permanent position and form when washed to place with water. Any subsequent consolidation is too slight to effect up-stream paving that has been properly detailed and laid on a 1 on 2½ slope.

(5) This available embankment material is deficient in fines for the usual impervious earth core designs, and must be supplemented by some impervious element forming a part of the embankment. Alkali treatment should be investigated.

(6) Embankments of this material, on good sand and gravel foundation, can probably be constructed satisfactorily for heads in excess of 100 ft, by re-arranging the foundation cut-off and impervious members to better structural advantage, and with improved economy.

(7) Penstock structures properly designed in articulated sections for settlement may be built safely through embankments and on foundations as herein described.

DISCUSSION

Charles W. Sherman,[5] M. Am. Soc. C. E. (by letter).—The statement under "Design and Settlement of Embankments" that, for an embankment of pervious sand, the plane of saturation generally "remains below a 1 on 5 slope from head-water to tail-water," provides a convenient approximation, but does not go far enough to be of particular significance. Referring to this slope as a "percolation factor," Mr. Burd states that "finer materials require higher factors," that is, flatter slopes of the surface of saturation. Obviously, this is incorrect unless drainage is not provided in the down-stream part of the embankment; and obviously, too, the slope is a function of the quantity of water percolating through the embankment as well as the porosity of the earth of which it is built, not to mention the tightness of the cut-off wall. If an absolutely water-tight membrane were provided at the up-stream face, no water could penetrate the embankment; the percolation factor (as used by the author) would be zero at the membrane and infinity for the remainder of the distance to tail-water.

Four typical cases from the writer's experience show the following inclinations of the surface of saturation: 1 on 7.5 for a small dam built of coarse sand on a foundation of the same material, with an incomplete cut-off wall; 1 on 5 for a dam built of medium sand upon the same material, with a reasonably good cut-off; 1 on 2.6 for a dam of comparatively dense hardpan, and having a core wall about which little is known; 1 on 1.7 for a dam of dense hardpan, well compacted, but having no core wall, and provided with ample drainage at the toe of the slope. The quantities of water percolating were appreciable in the first case; noticeable but of no special significance in the second; too small to measure in the third (although the undrained toe was saturated); and impossible of detection in the fourth case. The line of saturation in the last example was determined by well pipes driven into the dam.

These comments do not in any way reflect upon the interest and value of the paper in so far as it relates to the practicability of building a high and reasonably tight dam upon pervious foundations.

F. B. Marsh,[6] M. Am. Soc. C. E. (by letter).—In experienced hands there is no reason why structures like those described should not be both feasible and safe. However, they do call for more thorough exploration and understanding of the underground conditions than would be the case if the cut-off were being carried down to rock. Safety depends primarily on the adequacy of the drainage system and its uncloggable effectiveness under all parts of the down-stream toe, whatever the variations in foundation conditions.

In the last few pages of a paper[7] describing the Scituate Dam of the Providence, R. I., water system, the writer suggested the virtues of locating the cut-off at the up-stream toe. Time and money can be saved by using steel sheeting for the cut-off, rather than trying to make it completely watertight, as long as ample provision is made in the down-stream toe for draining all seepage water. Having been nurtured in a conservative eastern environment and being already classed by some as a heretic, the writer had not dared suggest that there might be locations where such a design was feasible even if the rock was too deep to be reached by the cut-off. Now, it has actually been demonstrated. That is progress.

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[5] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[6] Baltimore, Md.

[7] *Journal*, New England Water Works Assoc., Vol. XL (1926), p. 477.

There are many shades of conservatism. For example, there are those who still hesitate to rely on a line of steel sheeting for a cut-off under an important dam, even where the sheeting can be driven to rock. For the consideration of such persons the writer would like to suggest a double line of steel sheeting, say, 20 ft apart, starting either from the original surface, or from the bottom of an open trench of any convenient depth. By providing a sump between the two lines of sheeting, it would then be possible to lower the ground-water in this area and, by measuring the pumpage, obtain an approximate idea of the tightness of the sheeting. Grout pipes could then be put down outside both lines of sheeting, and continued pumping would tend to draw the grout through the points where the sheeting was not tight to the rock. The amount of pumpage should give a fairly direct measure of the effectiveness of the grouting, which could be repeated as often as necessary. In a long structure, such a cut-off could be left with provision for several cross-lines of sheeting, which could be driven later to divide it into bays to be tested separately as desired.

There are many locations where deep cut-offs of steel sheeting are impracticable, because of the presence of boulders or large beds of coarse gravel. Where conditions are favorable to driving steel sheeting it certainly offers economies in cost of construction. If any considerable depth is to be penetrated, the steel section will have to be heavy to withstand the driving operations. With a substantial steel section there seems no question as to the permanence of the cut-off. Even if ultimately, the steel should rust through entirely, the rust would occupy a greater space than the original steel and would still be effective as a cut-off.

M. M. O'SHAUGHNESSY,[8] M. AM. SOC. C. E. (by letter).—This paper is very illuminating with regard to problems in dam construction in the Michigan glacial drift formation. The most interesting part is the introduction of a concrete core into the hydraulic fill, with precautions taken to make the core tight and impervious. Further information concerning the details of the design and construction of this core-wall would be interesting. Where water does not have very great value, a leakage of 3.7 sec-ft is not excessive, provided it does not impair the stability of the dam structure if a substantial volume of water is retained behind the dam.

In Western communities this leakage would be regarded as excessive. Its existence might impair the safety of the structure. For example, in the Priest Dam of the Hetch Hetchy Project, 1 160 ft long and $147\frac{1}{2}$ ft high, the leakage amounts to only 219 600 gal in 24 hr, equivalent to 0.34 sec-ft, and has remained practically constant at that amount for the eight years, since 1925, when the dam was completed. The core-wall was constructed absolutely vertical by having the fills on each side kept at the same height.

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[8] Cons. Engr., San Francisco Public Utilities Comm., San Francisco, Calif.

The writer does not concur with Mr. Burd's contention that in constructing the Hardy Dam the use of floating equipment, giving access to fines, was undesirable (see heading, "Design and Settlement of Embankments"). The hydraulic method of handling available materials is certainly the most desirable from a cost point of view.

The drainage system on the down-stream base of the Hardy Dam is an excellent one. Its installation was a very admirable precaution. The paper as a whole opens up an interesting field in the economics of dam construction. The writer concurs in all the "Conclusions," except Conclusion (3). The settlement of the up-stream fill of Priest Dam of 2½ ft, and of the down-stream fill of 6 ft, confutes this statement. The construction of Priest Dam, 145 ft high, makes Conclusion (6) tenable. Excellent care devoted to the construction of the foundations of the core-wall is very essential to the completion of a satisfactory dam.

H. de B. Parsons,[9] M. Am. Soc. C. E. (by letter).—This is an interesting paper on the construction of dams on pervious foundations, and it contains information which should be useful to engineers who may have to erect structures under similar conditions. In the beginning of the paper, Mr. Burd speaks of the unusually uniform flow of the rivers in the glacial drift in Michigan and draws the conclusion that the hydro-electric field is reduced to supplying peak power. The conditions described are also favorable for the hydraulic plants to supply prime power and for letting the steam plants take the peak loads.

Mr. Burd calls attention to layers of glacial silt and rock flour compacted into solid masses, which he terms "mudstone." Similar material was found in the Hudson River when excavating the foundations of the Spier Falls Dam.[10] The rock dipped sharply on both banks. This V-shaped pocket, about 400 ft wide at the river surface, was approximately 57 ft in depth. The upper portion was filled with coarse sand and boulders, then fine sand, and, at the bottom, very fine dark silt and rock flour, compacted into a solid mass that required blasting. This material was practically impervious, but crumbled into fine dust on exposure to light and air. Its supporting power was very high in its natural state, while buried and held by the geologic rock floor. Practically no boulders were found in this mass. It was considered treacherous and was taken out in order that the base masonry of the dam could rest on the rock.

Mr. Burd refers to the settlement of the foundations in Michigan as being proportional to the load, but the writer's experience has been that true proportional settlement cannot be predicted. Mr. Burd also speaks of fine sand grading down to quicksand, but the writer does not like the word, "quicksand," as there are fine sands which are not "quick." The material was apparently fine sand with "quick behavior," and is always treacherous. The writer acted as Consulting Engineer for the construction of a power house that had to be founded on a bed of sand beneath which was a layer of sand with quick behavior. The work was accomplished by confining the material by steel interlocking sheet-piles, so as to divide the area into rectangles about 41 by 35 ft. These rectangles were excavated alternately until the sand blanket was reached, but the lower layer of sand with quick behavior (which had an artesian pressure) was left undisturbed. Occasional boils formed which were plugged by pouring broken stone (concrete coarse aggregate) into them. The broken stone acted as a reverse filter and soon clogged itself with the finer material brought up by the artesian flow. Such foundations are expensive, but at times cannot be avoided.

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[9] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

[10] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1326, Fig. 31.

Mr. Burd refers to paving the up-stream face of sand dams with concrete. Concrete facing has not always proved water-tight. The writer had an experience with a canal which was so lined, but when the canal was rapidly dewatered, the hydrostatic pressure on the outside burst the canal lining inward. The leakage, through the concrete lining, remained in the natural sand beds through which the canal was dug. In other words, the drainage through the sand was not rapid enough to remove the head due to leakage. The writer also understands that some rock-fill dams, faced with concrete, have not remained tight, probably due to settlement and shrinkage under their water loads.

JOEL D. JUSTIN,[11] M. AM. SOC. C. E. (by letter).—The old textbooks taught that earth dams should be built only of impervious materials on foundations of like materials, that masonry core-walls reaching to bed-rock were, to say the least, highly desirable, and that concrete spillways and power houses should never be built except on solid ledge rock.

This idea has become so thoroughly engrained in the minds of some engineers that they have refused to consider the feasibility of dam sites where such conditions were impracticable of attainment and, in other cases, have insisted on the expenditure of vast sums to secure such conditions. With regard to hydro-electric developments this attitude has been responsible for the rejection of projects that would have proved economic if designed and built on the principles followed in Michigan and described by Mr. Burd. In fact, the successful functioning of numerous hydraulic structures on pervious foundations in Michigan and elsewhere is proof positive that it is not necessary to have the ideal conditions specified by the old textbooks in order to construct successful dams of concrete or earth. As conclusively shown by Mr. Burd, it is merely necessary that all the pertinent conditions be analyzed and understood in order to be able to design and build successful hydraulic structures on pervious foundations.

The author states that "the advance of steam generation has reduced the hydro-electric field to that of supplying peak power." Throughout considerable sections of the United States this statement is measurably correct. However, the restriction is a broad one and leaves ample room for economic hydro-electric development to an extent exceeding past experience. Statistics show that the utilization of water power relative to steam is not decreasing but that it has been increasing in the past few years. In the year ending June 30, 1933, 41.4% of all the electrical energy generated in central stations of the United States was from hydro-electric plants, as compared to 35.9% for the previous year. In 1927, of all installation in central stations of the United States, 26.0% was hydro-electric and, in 1931, this had increased to 28.3 per cent. The average annual capacity factor for all central station steam plants in the United States for the year ending June 30, 1933, was 21.1%; whereas, the corresponding value for central station hydro-electric plants was 39.8 per cent. These facts are interesting, particularly as they are quite contrary to what is frequently assumed to be the case.

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[11] Cons. Engr., Philadelphia, Pa.

The frequent superiority of hydro-electric power for peak-load service is just beginning to be recognized by public utility executives.[12] Primarily, this superiority arises from the fact that the unit incremental cost of hydro-electric installation is low; that is, the cost per kilowat of installation of intake, conduit, power house, and hydraulic and electrical equipment, etc. In many cases, such incremental cost for hydro-electric projects with ample weekly pondage varies from $50 to $75 per kw. If the relation between minimum stream flow at the time of system peak load and the system peak load is such as to permit such hydro-electric capacity to operate when required, it performs the same service as that done by the ancient steam plants which are retained in service for the purpose of supplying the upper portion of the annual peak load.

The ancient steam plant requires a considerable crew to maintain it and keep it ready to operate when required and even if no useful energy is produced, a considerable quantity of coal must be burned. The annual unit cost of such old steam plants, excluding any return on the investment, varies from $8 to $15 per kw per yr, without the generation of any useful energy. On the other hand, maintenance and operating costs on the alternative hydro-electric units are insignificant and the total annual cost, including fixed charges on the investment, may be from $5 to $8 per kw per yr on an incremental basis.

In any proposed plan for superseding old steam capacity by hydro-electric units, it is essential, of course, first to make sure that the hydro-electric units can perform the same functions as those required of the superseded steam plant. Hydro-electric plants with ample reservoirs and pumped storage hydro-electric plants are both promising possibilities for peak load and reserve service.

Mr. Burd states that, with the embankment materials utilized in such structures as the Hardy Dam in Michigan, a percolation factor of 5 is desirable. This means that the shortest distance that water could travel from the reservoir to tail-water, or to the down-stream side of the embankment, should be not less than five times the head. He then proceeds to state that for finer materials the percolation factor should be greater. In other words, the slope of the line of saturation would be flatter if finer materials were used. Such a statement requires qualification as it is not necessarily true. As has been pointed out by the writer,[13] the slope of the line of saturation is determined not only by the size of the particles composing the embankment but also by (1) the particles composing the foundation material; (2) the porosity of both foundation and embankment; (3) depth below base of dam to rock or other impervious material; (4) ground-water conditions; (5) temperature; and, last but not least, (6) provisions made for drainage.

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[12] This subject is treated at some length in "Power Supply Economics," by Joel D. Justin and W. G. Mervine, John Wiley & Sons, Inc., 1933.

Consider a dam built of materials finer than those used in the Hardy Dam and in which the materials in the foundation have the same degree of fineness as those used in the embankment, and the same porosity; and that the natural ground-water is at the same elevation as it was at the site of the Hardy Dam. Then, the line of saturation would be at a slope much flatter than that which occurs with the coarser materials at the Hardy Dam site. The percolation factor would thus be higher. This is in accord with the statement made by the author.

On the other hand, if the embankment were built of much finer materials than those used at the Hardy Dam, but the foundation material was coarse, the slope of the line of saturation in the dam might be much steeper than was the case at the Hardy Dam and, consequently, the percolation factor for the dam would be less. Artificial drainage in a dam of finer material might also cause the percolation factor to be smaller.

Fine embankment materials do not necessarily require flatter slopes than those used at the Hardy Dam for coarse materials. Thus, many successful dams of clayish materials, built in rolled layers, have slopes as steep as those utilized at the Hardy Dam.

Many construction superintendents dislike a core-wall extremely. It usually interferes with the operations of embankment placing and it is claimed that it increases the total cost of the work to a greater extent than can be shown by any cost segregation. Apparently, the cost of the core-wall at the Hardy Dam was only about 15% of the total cost of the embankment. In some cases with which the writer is familiar, the cost of the core-wall, including the cut-off and excavation therefor, has been as much as 50% of the total cost of the embankment. The advisability of using a core-wall and the steel sheet-piling under it is largely a matter of economics. It has been the writer's experience that where a cut-off can readily reach ledge rock or an impervious layer, and where only relatively pervious material is available for the embankment, a core-wall of the Michigan type is frequently economical. Where there is no impervious layer to be reached, and where embankment and foundation materials are homogeneous, the writer has usually found it more economical to omit the core-wall and secure the necessary percolation distance by flattening the slopes of the dam, sometimes with the added precaution of an up-stream blanket of fine material.

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[13] "Earth Dam Projects," by Joel D. Justin, John Wiley & Sons, Inc.

From a theoretical standpoint, the idea of placing an impervious membrane on the up-stream face of an earth dam instead of using a core-wall has all the advantages which the author claims for it. Quite a few dams have been built with such a membrane, but the results have not always been satisfactory. A reinforced concrete core-wall, such as those commonly used in Michigan and elsewhere, is buried in the body of the dam and protected from the action of the elements. It is subject to only slight distortion, which it may be designed to take without losing its impervious character. If designed and constructed with moderate skill, it may be relied on to retain its relatively water-tight characteristics.

One cannot feel the same assurance with regard to a reinforced concrete diaphragm laid down on the inclined up-stream face of an earth dam. Settlement may cause distortion and, with the aid of temperature changes, cracking of the diaphragm; or, if concrete blocks are used, these may be displaced, opening cracks at the joints. In case the water in the reservoir is drawn down suddenly internal hydrostatic pressure in the embankment may cause rupture of the diaphragm. A relatively small crack or opening through the concrete will supply a very large area of the up-stream face of the embankment with as much water as it could absorb if entirely unprotected by the concrete diaphragm.

On the Owl Creek Dam, near Belle Fourche, S. Dak., during a period of high waves, the receding waves momentarily relieved the pressure on the concrete facing blocks, so that the internal hydrostatic pressure from the embankment forced out about two hundred and fifty of them.

In the construction of rock-fill dams, reinforced concrete diaphragms on the up-stream face, usually of flexible jointed blocks, are standard practice; but, here, the backing is 100% free draining and in case of a rapid drawdown of the reservoir, there is no chance of an unbalanced upward pressure.

Even with the sand dams of Michigan, it is conceivable that in case of a rapid drawdown of the reservoir, the water in the dam might be at a higher level than the water in the reservoir. The difficulty might be overcome by using a heavy fill of coarse material and rock on top of the inclined reinforced concrete paving. The load placed on the paving should be great enough to counterbalance any upward hydrostatic pressure which might come on the bottom of the concrete slab as a result of a rapid drawdown. To insure that the fill on top of the concrete was not displaced by wave action, it would have to be protected by rip-rap or large stone. In many cases, such requirements would take away all the savings claimed for the plan.

Concrete lining has been used to a large extent on the water slopes of the distribution reservoirs of water supply systems. The function in this case, however, is usually not to secure water-tightness, but to obtain a smooth hard surface to facilitate cleaning and also to protect the embankment against wave action. Concrete blocks, often laid on a bed of gravel, also have been used frequently on the up-stream face of high earth dams to provide protection against waves. In such cases it has usually been found desirable to provide weep-holes or other spaces through the concrete blocks so that when the water in the dam is at a higher level than the water in the reservoir, it can seep out without causing an unbalanced hydrostatic pressure on the bottom of the concrete blocks.

A. K. POLLOCK,[14] ASSOC. M. AM. SOC. C. E. (by letter).—The work described in this excellent paper is perhaps rather bold and daring in its conception, but the result would appear to have justified the designer's optimism. It is not novel to build a dam on a sand foundation, but the height of the Hardy Dam makes it a notable structure, especially when it is remembered that the dam itself is also of sand.

Bligh has given[15] comprehensive data on the construction of weirs on sand foundations, and Strange developed[16] a type of earth dam for use where percolation under and through the dam was unavoidable. The chief feature of this type is the extensive system of under-drainage.

The percolation factor of 4.25 used at the Hardy Dam is lower than any advocated by Bligh, who gives a factor of 12 for coarse sand and a range of 9 to 5 for shingle, gravel, and sand mixed. Mr. Burd states that "experience has indicated a safe generalization for this form and composition of embankment, namely, this plane [of saturation] remains below a 1 on 5 slope from head-water to tail-water." From Fig. 5 the material at the Hardy Dam would be classed as coarse sand. The author's experience, therefore, would seem to indicate that Bligh's factors are rather on the conservative side.

A curious feature of the Hardy Dam and others of its type, is the practical cessation of settlement on the completion of construction. Sand is virtually incompressible, of course, and most of the settlement that takes place is due no doubt to the packing and arranging of the grains of sand during the washing-in process. A point not mentioned in the paper, is the effect on the dam of the bulking property of sand. It would be interesting if the author could state what, if any, were the observed effects of the decreasing moisture content of the dam as the water used in construction drained away. Experiments in connection with the measurement of sand for batching concrete have shown that saturated sand has approximately the same volume as dry sand, but that when the moisture content is in the order of, say, 5 to 10%, the sand increases in volume perhaps as much as 20 per cent.

During the construction period of the Hardy Dam, it is assumed that the water used for washing the sand into place was sufficient to saturate the material which would then occupy its minimum volume. As the water drained away, the percentage moisture content must have dropped to a point at which measurable bulking would occur. How did this affect the settlement records? When the reservoir was filled, that part of the embankment below the line of saturation would revert to the condition of minimum volume, but the part above the line of saturation would probably reach a condition where the moisture content was such as to cause maximum "bulking," and this part of the bank, therefore, would be in a condition of maximum volume. Observations of settlement on stakes placed on the down-stream slope would be difficult of correct interpretation if this "bulking" did occur. If there was no real settlement a stake would rise with the bulking of the embankment. If the peg showed no movement, may not this be interpreted to mean that the bulking of the material counteracted the settlement?

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[14] Care, The Water and Sewerage Board, Kingston, Jamaica.

[15] "Dams and Weirs," by W. G. Bligh, 1927.

[16] "Indian Storage Reservoirs with Earthen Dams," by W. L. Strange, Third Edition, 1928.

The foundation settlement appears to have been observed only by reference points on the core-wall, power house, and penstocks. Of course, this does not give the settlement of the valley bottom due to the superimposed weight of the dam, and it would be interesting if Mr. Burd could give any information on this point. Due to the core-wall being founded on sheet-piling, there would be a considerable differential settlement between the core-wall and the valley floor, on either side of it.

William P. Creager,[17] M. Am. Soc. C. E. (by letter).—A partial comparison of the Hardy Dam with the Soft Maple Dam is included in this excellent paper. It is of interest to make further comparisons since the two dams are similar in many respects. They have practically the same height and thickness, but the Hardy Dam is 2.7 times as long as the Soft Maple Dam. They are built of practically the same materials and, with the exception of the mudstone layers, the foundations are almost the same.

The chief source of difference lies in the use of a core-wall and up-stream blanket for the Hardy Dam and the mudstone layers in its foundation. The author gives the following data for seepage at the Hardy Dam:

	Initial	After six months
From the pond	3.7	. .
From ground-water	3.0	. .
Total	6.7	5.7

The seepage at the Soft Maple Dam is:

	Initial	After 1 year	After 8 years
From the toe of the dam	8	6	4.5
From the adjacent hill	6	4	3.0
Total	14	10	7.5

It will be noted that the seepage from the Soft Maple Dam is about twice that from the Hardy Dam despite its much shorter length. How much of this difference is due to the core-wall, to the up-stream apron, or to the mudstone layers in the foundation, is problematical. It will be noted that a large part of the seepage from the Soft Maple Dam came from springs in the adjacent hills, which seepage would be little, if any, affected by changes in the design of the dam. There is also evidence at the Soft Maple Dam that an appreciable quantity of the water seeping from the toe passes around the dam, the piezometers at equal distance from the top of the dam showing higher gradients near the ends than in the middle. In all probability not more than 25% of the total seepage is passing through the dam.

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[17] Chf. Engr., The Power Corporation of New York, Buffalo, N. Y.

The effect of a core-wall in a dam on an equally or more pervious foundation is problematical. The writer made some experiments to determine the effectiveness of a cut-off under an impervious dam on a deep pervious foundation overlying rock. It was found that, for a cut-off extending 40% to rock, the seepage was reduced only 23% and even with a cut-off extending 82% to rock, the seepage was reduced only 50 per cent.

Therefore, a cut-off that extends only a relative short distance into a deep homogeneous pervious foundation has little additional effect on seepage compared with a core-wall that extends only to the surface. It increases the resistance to flow under and around the dam and, therefore, the slope of the hydraulic gradient passing the core-wall by virtue only of a slight reduction of area and a slight increase in percolation distance.

A core-wall at Soft Maple Dam, extending a practical distance into the foundation, would have reduced the total seepage certainly not more than 10 or 15 per cent. A blanket of fine material on the up-stream face and extending a considerable distance up the bottom and sides of the valley, would have been much more effective. The presence of mudstone layers at the Hardy Dam, if sufficiently extensive, into which the core-wall penetrates, may have had a considerable bearing on the lesser seepage.

ADOLPH J. ACKERMAN,[18] ASSOC. M. AM. SOC. C. E. (by letter).—The excellent paper by Mr. Burd is one of the first comprehensive records of an important phase of engineering which has been in the process of development during the past thirty years. The contributions by Michigan engineers to the field of dam design have remained comparatively unknown, and many of their well-established and sound principles, described in this paper, will prove surprisingly new and bold to others.

The author's use of the term, "sand dam," deserves particular emphasis in distinguishing it from the more frequently and loosely used term, "earth dam." Its distinctive characteristics are evident from inspection of Fig. 5 which shows the isolated position of the mechanical analysis curves with respect to those of more typical earth dams.

Fig. 7 is offered as a convenient basis for classifying the various types of natural-fill dams. It takes into account the full range of natural materials. Since the type of construction procedure, rolled, hydraulic, or semi-hydraulic (terms which are often used to designate the type of dam), is dependent chiefly on local and economic factors and, to a lesser extent, on the type of material, it is suggested that such designations, when used in combination with the terms on the curve, will be more descriptive and positive in definition. Where several materials are involved, the practice of some engineers to use a combination name, such as "earth-gravel-rock-fill dam," is obviously more satisfactory than merely "rolled and dump-fill dam."

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[18] Constr. Plant Engr., Tennessee Val. Authority, Knoxville, Tenn.

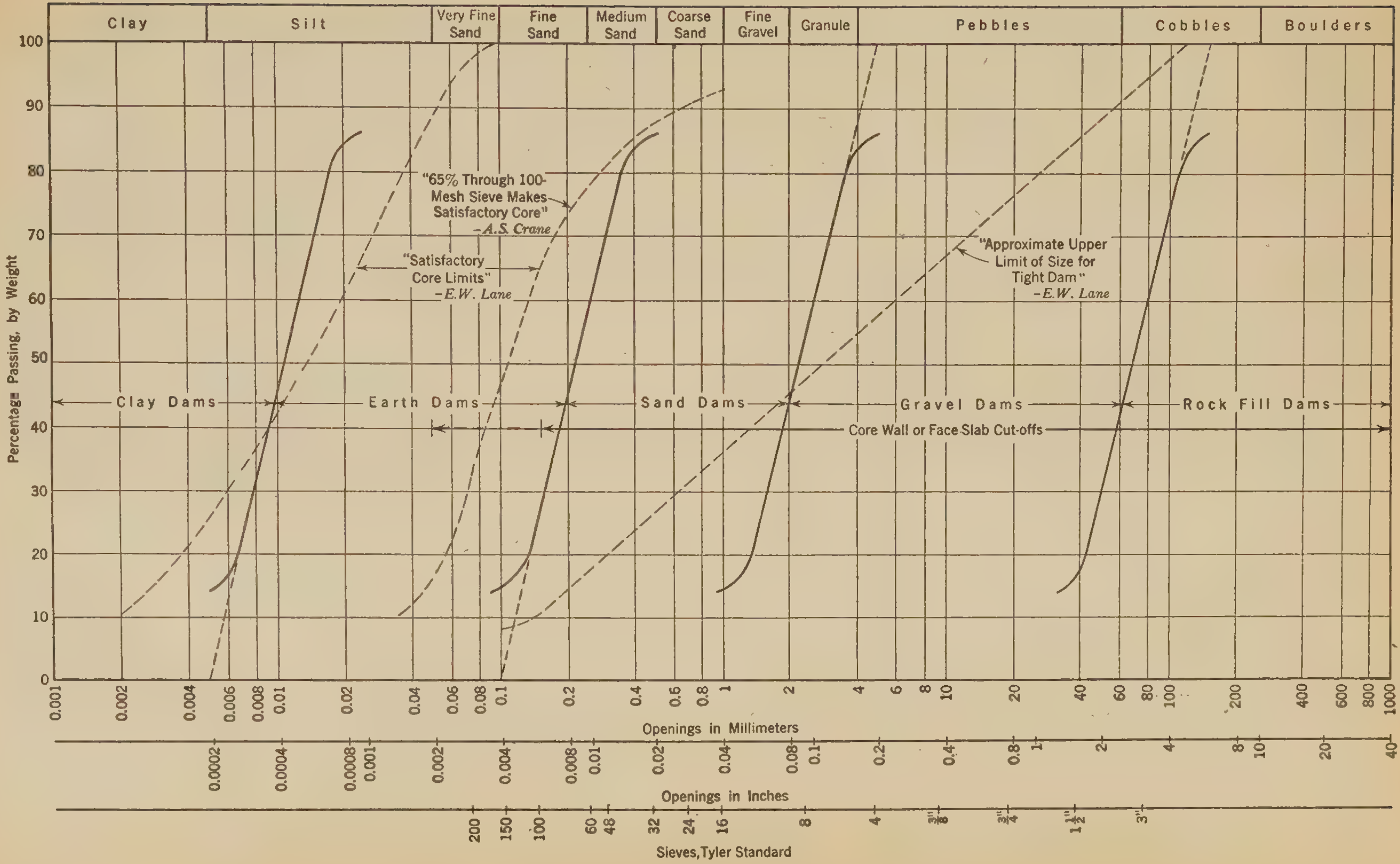

FIG. 7.—CLASSIFICATION CHART, NATURAL FILL DAMS

While a classification chart based on mechanical analyses has proved satisfactory for comparing materials of various dams, it must be repeated that the suitability of a material for a proposed dam is not defined thereby. The factors of solubility, "sloughing when saturated," ease of compacting, "sticking to roller," failure to consolidate rapidly, permeability, pumping limits, etc., depend upon other tests.

A number of controversies have been staged over the utility of core-walls in earth dams, and there are, unquestionably, several dams in which an abundance of impervious core material has made the adoption of a core-wall appear unduly conservative. The author has not only demonstrated the merits of a core-wall for the special conditions of his region, but has properly concluded that in a sand dam percolation characteristics are such as to require the same treatment of design that is applied to gravel dams, such as the Mackay Dam, at Pendleton, Ore., or, to a limited degree, to rock-fill dams; his recommendation, therefore, to place the cut-off diaphragm on the face of the sand dam is obviously logical and is certain to result in a more economical structure, as long as adequate safeguards have been taken in the design of the face slab to maintain its water-tightness.

The writer was employed from 1923 to 1925 on two of the projects under discussion, and recognizes the very practical features in the proposed improved sand embankment (Fig. 6). The proposal to sluice the material up stream on a slope of 1 on 8 is amply conservative, since measurements taken under similar conditions, where the material was allowed to run parallel to the axis of the dam, gave a slope of 1 on 20.

Where the intake and power house are combined in a single box-like structure and serve as a section of the dam, the requirement of approximately uniform base pressure under full head, instead of the customary middle-third theory of stability, places special demands on the designer's ingenuity. As a rule, the base slab and intake walls are tied together with reinforcement to utilize the weight of the superimposed water, and parts of the structure not required for water passages are designed as cells and filled with sand.

On the project built in an area of quicksand, the writer was confronted with the perplexing fact that with the addition of a new lift of concrete the beginning of settlement invariably lagged several days behind the pouring dates. A satisfactory explanation has since been offered[19] by Professor Terzaghi who showed that in saturated foundations this time lag represents the mechanical action of squeezing out the water.

The intake tower described in the paper as a monolithic structure is of more or less standard type, but the foundation problems assume special importance when ice thrust is considered, and, no doubt, some additional details on this point would be of general interest.

J. ALBERT HOLMES,[20] M. AM. SOC. C. E. (by letter).—About thirty years ago the members of a Board of Engineers averred they could find no precedent for building an earth dam, with core-wall, to a height of 150 ft, but that to a height of 70 ft an earth dam on a firm foundation, properly built of selected material, was as safe and lasting as a masonry dam.

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[19] "The Science of Foundations—Its Present and Future," by Charles Terzaghi, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 278.

[20] Cons. Engr., Chicago, Ill.

The Board was mildly taken to task for following precedent so closely, and for forgetting that the manifest destiny of the engineer is to control and direct the forces of Nature for the benefit of Mankind. In this case, Mankind would have been greatly benefited, in his pocket, had the Board for the moment forgotten precedent and forced Nature to disclose a method of building a safe earth dam to a height greater than 70 ft. Precedent need not have been abandoned, however, because wider investigation would have disclosed the fact that several earth dams were in existence exceeding 100 ft in height and many more than 70 ft high. Two of the high ones, 120 ft and 125 ft, both without core-walls, were in California. Strangely enough, no technical description of any of these dams could have been found at the time. Engineering literature had "slipped," perhaps had not recognized the significance of dams of earth, although that was the material most anciently and continuously used in building dams and dikes. Quickly there came a flood of information, filling engineering periodicals and overflowing into books. By doubting, the Board of Engineers had served the art of earth-dam building a good turn; but dams of sand were not within its purview.

Clay is naturally the most impermeable of earthy materials, but it is not found everywhere in the world, and Michigan is one of the regions where it is not always available, from which to "select the best." Sand is available, however, and has been used there and elsewhere as a dam-building material. Its successful use is a demonstration of the "control and direction of the forces of Nature."

British engineers in India, obliged to make use of sand, the material at hand, have determined in advance the slope that percolating water would take in passing through that particular sand, and then have given the down-stream face the same or a flatter slope, helped out at the toe by an inverted filter, ending in heavy rip-rap.

The designers and builders of Hardy Dam, by experience and research, were thoroughly familiar with the characteristics of the construction material they must use. Their problem was the economic control of seeping water. The solution was to construct a flexible cut-off wall, supplemented by deep sheeting below, assisted by a rock-filled trestle and pipe drains at the down-stream toe.

Granular materials, like sands, are relatively incompressible when confined. If the grains can shift their positions, considerable change in volume can take place, as when a loose sand is compacted by ramming or shaking; but without the possibility of such a change of position, the volume of the mass changes very little under pressure. A few pounds of air pressure applied to a mass of sand in a rubber container will render the material as firm as a rock, thus demonstrating the ability of undisturbed sands to sustain heavy loads.

The foundation materials under the valley floor were restrained from lateral motion, and it is not surprising that the settlements at Hardy Dam were so slight. Heavy loading of an area would cause horizontal bulging in the direction of less heavily loaded areas, but this was foreseen by the designers and care was taken that there should be no abrupt change in pressure, that is, changes that would cause shear and the opening of waterways in the foundation material.

Among the mechanical analysis curves in Fig. 5, those for the Hardy Dam and for the Bridgewater borrow-pit materials lie close together, especially in their lower sections, in the smaller sizes. Mr. Burd thinks that this might indicate that sluicing at Hardy Dam would have produced a core as impervious as those in the Bridgewater Dams. This is an illustration of the varying characteristics of materials that must be used in the construction of dams in different parts of North America. The author has described the Michigan dune sands of that glaciated region. Bridgewater, N. C., lies south of and beyond the advance of the ice sheets. The writer has described the geology of this region and the borrow-pit material in some detail in a previous paper.[21]

In the pit face this material still retains the markings and appearance of the original ledge, but can be excavated readily by steam shovel without blasting; it breaks up further in handling. The cores of the three dams sluiced from this material have effective sizes of 0.026, 0.028, and 0.030 mm. The grains of the Bridgewater materials are not hard like those of the glacial sands of Michigan, and under pressure in place in the dams, and under the pressure applied in connection with the seepage tests, may have broken down in part and produced smaller sizes and a more dense material in cores and test tank than the aforementioned sizes, determined from unpressed samples, would imply.

A sample of Michigan dune sand has given an effective size of 0.218 mm. The effective size of sand grains in Soft Maple Dam was about 0.17 mm.; but what is more indicative of porosity of a material is its uniformity ratio. The uniformity ratio of the dune sand was 1.65, indicating high porosity.

As an alternative method of construction Mr. Burd suggests the use of a single set of dumping trestles in the down-stream embankment and sluicing to the up-stream face. Sand and gravel, containing considerable fine material, can be controlled successfully on steep slopes by means of shear-boards or their equivalents. During the period, 1895 to 1897, the Northern Pacific Railroad Company placed, by sluicing, more than 600 000 cu yd of fill in fifteen trestles in the State of Washington, at an average cost of 6.39 cents per cu yd. The purpose of making the fills was to eliminate the trestles. The slopes approached more nearly those of railroad fills than the slopes of dams. The material was conveyed from pits to fills by ground-sluicing and sluice-boxes; no material was dumped from trestles.[22]

While the sand and gravel could be held on the slope, the author suggests, it might not sluice on grades as flat as 1 on 8 without follow-up methods; that is, without using water under pressure at considerable distances from the trestles. At Wanaque Dam, in New Jersey, the sand and gravel, dumped from belt conveyors on the cut-off wall, was sluiced to place, but did not assume very flat slopes, except the smaller sizes.

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[21] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 332.

[22] "Reservoirs for Irrigation, Water Power, and Domestic Water Supply," by the late James Dix Schuyler, M. Am. Soc. C. E., Second Edition, 1909.

The importance of consolidating sand and gravel fills with water cannot be too strongly emphasized. No other method will arrange the grains in large masses in so compact a manner as water. Even standing water, as by irrigation, will bring about remarkable compacting. Caleb M. Saville, M. Am. Soc. C. E., has demonstrated[23] what may occur, in the two methods of sluicing, to the up-stream face, or to the center of a dam. The line of saturation is always above or coincident with the line of complete saturation; that is, the line shown by standing water in bore-pipes or model gauges, unlike the hydraulic grade, begins at the junction of the dam face and pond-water surface. At times, especially at the toe, it is higher above complete saturation than the capillarity of the material. The line of complete saturation and the line of capillary saturation are both of interest and importance, but if and when used (and for clarity), they should be designated fully and correctly and not confused with or called the hydraulic grade.

From tail-water at the rock-filled trestle to pond-water surface at the splash wall (Fig. 1), the slope is about 1 on 2, or approximately parallel with the upper part of the embankment slope. A saturation line is shown on the cross-section of the dam (Fig. 1), apparently drawn from the top of the rock-filled trestle to the cut-off wall on a slope of 1 on 5. This slope is an average, determined by observation and the experience gained by the designers from other structures of similar materials built by them, and lies safely within the lines of the down-stream embankment. Its position depends on the integrity of the cut-off wall and the ample drainage afforded by the rock-filled trestle, and they, in turn, make possible the steep slopes and economic section of the down-stream embankment.

The writer's understanding of the "saturation line" and its position, as shown in Fig. 1, and on the proposed dam section having a paved up-stream slope (Fig. 6), is that with the drainage provided at the toe, saturation might take place to that line, at the ends of the dam, from hillside seepage and elsewhere, in case of leakage through the cut-off or paving. This type of flexible cut-off was developed previous to 1908 by Mr. Fargo, as mentioned in the paper.

In the absence of a mudstone layer with which to connect cut-off sheeting at the up-stream toe (Fig. 6) an embarrassing quantity of seeping water might be forced up and into the coarser materials in the down-stream part of the embankment. Modern laboratory investigation of the influence that cut-off walls and their positions have on flow under sand dams, would furnish valuable information.

The author's curve of total settlement of the crest wall (Fig. 4), shows, by a series of four dots just below the 3-in. line, a period of rest at the end of the embankment building period and the beginning of pond filling. This is a characteristic record of embankment settlement and the writer would expostulate with the author for not permitting his curve to follow the dotted line.

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[23] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 97, 39 and 40.

Foundation material does not settle appreciably until a considerable load has been applied. It is then rapid during loading and less rapid thereafter. The greatest percentage of settlement takes place at the base of the fill. To measure the settlement taking place during construction and at various depths, a series of plates and attached pipes were placed in the rolled fill embankment of Fifteen-Mile Falls Dam, on the Connecticut River, at Barnet, Vt.

The embankment is a clay and gravel structure, Fig. 8, having a maximum height of 162 ft and abutting against a high retaining wall at the foot of the east bluff. The space between the back of the wall and the foot of the bluff was narrow. Here, the boulder clay fill was placed by hand and air-tamped. The remainder of the fill, to the top of and on the bluff, was delivered in trucks, spread by "bull-dozers" and rolled in layers not less than 6 in. thick after rolling. The first plate and pipe, No. 1, was placed on the layer of air-tamped clay, 18 ft thick, at that point. Other plates and pipes were placed at 30-ft intervals in the clay fill, and immediately after the completion of the fill six plates with attached rods were placed in the top of the embankment. Well-points were attached to the deep plates and surrounded by a gravel-filled steel frame. This was for determining water elevations in the fill. Elevations of plates were obtained as each 5-ft section of pipe was added. Frost penetrates deeply in the latitude of Fifteen-Mile Falls and to keep below the frost line 7-ft rods were used with the plates placed in the finished embankment. For the protection of the rod and to prevent gripping and lifting by the frozen ground, a pipe is placed with the rod. The grease in the pipe excludes surface water, which in freezing would disturb the rod and plate.

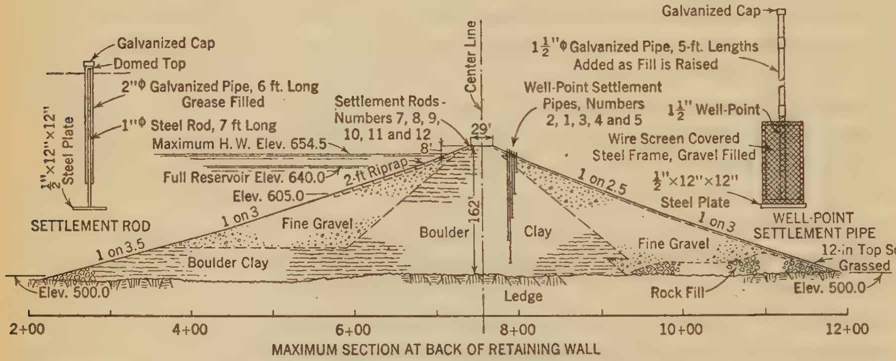
Galvanized Cap
Domed Top
2"Φ Galvanized Pipe, 6 ft. Long Grease Filled
1"Φ Steel Rod, 7 ft Long
$\frac{1}{2}$"×12"×12" Steel Plate
SETTLEMENT ROD
Settlement Rods - Numbers 7, 8, 9, 10, 11 and 12
Maximum H. W. Elev. 654.5
Full Reservoir Elev. 640.0
2-ft Riprap
Elev. 605.0
8'
29'
Center Line
162'
1 on 3
1 on 3.5
Fine Gravel
Boulder Clay
Boulder
Elev. 500.0
Ledge
Galvanized Cap
1$\frac{1}{2}$"Φ Galvanized Pipe, 5-ft. Lengths Added as Fill is Raised
Well-Point Settlement Pipes, Numbers 2, 1, 3, 4 and 5
1$\frac{1}{2}$" Well-Point
Wire Screen Covered Steel Frame, Gravel Filled
1 on 2.5
$\frac{1}{2}$"×12"×12" Steel Plate
1 on 3
WELL-POINT SETTLEMENT PIPE
Clay
Fine Gravel
12-in Top Soil, Grassed
Rock Fill
Elev. 500.0
2+00
4+00
6+00
8+00
10+00
12+00
MAXIMUM SECTION AT BACK OF RETAINING WALL

FIG. 8.—ROD AND WELL-POINT SETTLEMENT PLATES AND PIPES, FIFTEEN-MILE FALLS, LOWER DEVELOPMENT

Experience at Fifteen-Mile Falls Dam demonstrated that total settlement in a well rolled embankment is not great. It occurs mainly during construction and is not recorded by elevations taken at the top of a dam after completion.

Irving B. Crosby,[24] Affiliate, Am. Soc. C. E. (by letter).—Hardy Dam is on the Muskegon River where it passes between the great interlobate moraine, which extends up the western side of the Southern Peninsula of Michigan between the Muskegon and Manistee Rivers, and the combined Charlotte and Valparaiso moraines, which extend south from the Muskegon River, the first swinging to the east through Southern Michigan and the second swinging to the southwest around the southern end of Lake Michigan.

During the retreat of the ice sheet the interlobate moraine was formed between a lobe of ice which occupied the Lake Michigan Basin and a lobe to the east. The moraine consists partly of more or less sorted débris washed from the ice and partly of glacial till pushed up during a re-advance of the ice. The till is an heterogeneous mixture of rock-flour, sand, gravel, cobbles, and boulders; and, depending upon the quantity of fine material, it varies from a hard, dense, boulder clay to loose sand, gravel, and cobbles. On account of this origin the composition of the moraine is very complex.

At the dam site, the Muskegon River flows through a broad valley which is floored with glacial outwash consisting of sand and gravel. Beneath this outwash is a great thickness of glacial till which is generally sandy. The river flows along the northwest edge of the valley near the interlobate moraine, and the northwest end of the dam almost connects with the moraine. The sand plain extends for several miles south of the dam. Prior to the construction of the dam the river flowed in an inner valley or gorge about 100 ft deep, which cut through the outwash sand and gravels into the till. The dam is across this inner valley and connects with the till and outwash deposits at either end. Embankment material was generally taken from the outwash deposits, which are less heterogeneous than the till.

Hardy Dam rests directly on a layer of so-called "mudstone" which has a maximum thickness of 20 ft, but which apparently was thicker before it was eroded by the river. This "mudstone" is a very fine-grained sediment containing small shells, and, in its natural condition, is moderately hard and very dense and impervious. It dries out slowly and can be easily crumbled in the fingers in either a dry or a damp condition. This "mudstone" contains practically no clay and is not plastic. When dried and thoroughly crumbled, all but a small percentage will pass through a 200-mesh sieve. The coarser fraction consists of shells and fragments of shells and of fine sand, largely rounded grains of quartz. That part which passes through the 200-mesh sieve varies greatly in grain size, and the grains are mostly angular. It consists largely of minute fragments of calcite with a small percentage of quartz and feldspar. This "mudstone" resembles, in composition, marls that are found in adjacent parts of Michigan, and it appears probable that this material is a marl formed by the chemical or organic precipitation of calcium carbonate in swamps. The fact that peat was found associated with it supports this view. The quartz and feldspar presumably were washed into the swamp and mixed with the calcite. The marl is overlaid by glacial till, showing that after its formation in a swamp it was over-ridden by an advance of the ice and subjected to great pressure, which accounts for its hard, dense condition. The till immediately overlying the marl is hard, dense, and reddish, and is quite different from the loose sandy till of the moraines.

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[24] Cons. Geologist, Boston, Mass.

The history of these deposits is apparently as follows: During the melting back of the ice sheet which covered Michigan in the last glacial period, great quantities of sand were deposited, and then swamps and ponds developed, in which marl and peat were formed. These deposits indicate the presence of animal and vegetable life. Then the ice re-advanced over the marl and peat swamps and laid down a layer of boulder clay which was compacted under the weight of the ice, forming the hard red till. During the subsequent melting of the ice the sandy till of the interlobate moraine was formed, and, finally, sand and gravel were deposited in a temporary lake at a much higher level than the lake in which the marl was formed.

Mr. Burd has emphasized the heterogeneity and irregularity of these glacial deposits and states that any but the most general deductions beyond actual exploration are wholly unreliable. It is true that with these glacial deposits little detail can be forecast without sub-surface explorations, but the results of such explorations can be interpreted more accurately by one who is thoroughly familiar with the geology of this type of deposit.

One of the most important problems in connection with a dam on pervious glacial drift is to estimate the seepage that will occur under and around the dam. With such irregular deposits this is difficult, but reasonably satisfactory results can be obtained. Estimating the seepage, however, requires thorough and accurate sub-surface information, undisturbed and unwashed samples, careful tests, and, above all, a thorough understanding of this type of deposits. Accurate boring samples are important, and wash samples are worse than useless. It is absolutely essential to obtain a sample from which nothing has been removed or to which nothing has been added by washing. If possible, an undisturbed sample with the original structure and compactness should be secured. With the best of boring samples, however, estimating the seepage under and around the dam is a much more difficult problem than estimating seepage through the dam, since in the first case the arrangement and condition of materials are at the best only imperfectly known, while in the latter case the arrangement and condition of the materials are controlled and can be known accurately. Since even elaborate sub-surface exploration does not give full or accurate information the best results can be obtained by a geologist experienced in this kind of work and who is familiar with the arrangement of this type of deposit.

Conditions at the Hardy Dam contrast strongly with those generally found at dam sites in the eastern part of Canada and the glaciated region of the Northeastern United States. At the Hardy Dam, and in a large part of the Southern Peninsula of Michigan, bed-rock is buried deeply, and the glacial deposits are thick and are generally of a sandy character. In this region, the present drainage systems have absolutely no relation to the preglacial drainage systems and are, with few exceptions, entirely independent of bed-rock control. At the Hardy Dam, and at other dams on the Muskegon River, bed-rock is at such great depth that it could not in any way affect the design of a dam.

In the East, the bed-rock topography is irregular as a rule, and the thickness of the glacial deposits varies greatly within short distances. In the more mountainous regions the streams are usually in their pre-glacial valleys, but not in their old channels, which are often buried at considerable depth. In the regions of low relief, however, the streams often pass from one pre-glacial valley to another and have little relation to the pre-glacial drainage systems; but even in these cases there are usually exposures of bed-rock, and its location often plays an important part in planning a dam.

The geological problems are not by any means the same in all parts of the Southern Peninsula of Michigan, and the conditions at Hardy Dam are probably as complicated as any, since it is at the southern end of a great interlobate moraine where till and sorted materials were both deposited.

In adjacent areas there may be dam sites with much simpler conditions. A site on an extensive outwash plain may have continuous layers of sand, gravel, or clay in a more or less orderly arrangement, and a site directly in a moraine may have foundations of boulder clay which would be tight and would offer no serious problems. The type of dam site and the nature of the problems can easily be determined by a brief geological investigation, and the details can then be investigated by borings and, if necessary, by permeability tests on samples. From all this information the geologist can then forecast conditions and answer the practical questions that will arise.

HARRY H. HATCH,[25] M. AM. SOC. C. E. (by letter).—It may be inferred that the author is belittling, unintentionally, the importance of good rock foundations for earth dams. The fact that 100 ft of head against an embankment 120 ft high on a foundation of pervious glacial drift is considered an accomplishment, should in no way classify the Hardy Dam as a high earth dam. For really high earth dams it is still necessary to follow the Biblical injunction and the caution taught by past and present experience.

The problem is a question of permanency and the permissible quantity of seepage both from viewpoints of its economic value and its effect on the safety of the structure. With his wide experience with this kind of foundation and embankment, the author has evidently solved the problem to the satisfaction of his requirements. This solution, however, should not be considered as of general application, but should be restricted to the same type of foundation, similar embankment material, the same height of dam, the same head of water, and the same economic conditions; and the same care should be exercised in construction.

It has been shown that, approximately, seepage varies: Directly as the square of the effective size; directly as the sixth power of the percentage of voids of the core material; directly as the square of the head; and, inversely, as the path of percolation. At Hardy Dam the loss of water by seepage may not have any economic value, but the same quantity may become a big factor elsewhere, and with a higher head, or for other reasons, the increase in seepage may prove fatal to the structure.

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[25] Engr. in Chg., Cobble Mountain Reservoir, Springfield Water-Works, Blandford, Mass.

Any difference in settlement, whether in the foundation or in the embankment, involving the relative superimposed load, is due to the difference of the percentage of voids of the respective materials, and also to the difference in their moisture content. Under a given load there is a limiting consolidation or a minimum percentage of voids in any given material. This consolidation takes place very rapidly at the beginning of the application of the load; then very slowly. The time necessary for ultimate consolidation will depend on the quantity of water to be squeezed out during the re-arrangement of the material particles from initial to final percentage of voids, or during their adjustment to final structural equilibrium under the maximum load. This also depends on the path of percolation, head of water, and particle sizes.

It would be desirable if the author itemized the 30-cent cost per cu yd with maximum and general average yardage per working day, analyzing the cost per yard in terms of: (*a*) Excavation; (*b*) conveyance to the dam site; and (*c*) placing in the dam.

It would be interesting to know how the author determined the position of the fineness gradation lines in Fig. 6. For instance, the 1.00-mm line which shows that almost 100% of the sluiced material is larger than 1.00 mm at the bottom of the embankment, and, at the bottom of the top trestle, it shows that almost 100% of the sluiced material is smaller than 1.00 mm. The intersection of the 1-mm line with the slopes of the sluiced material at various elevations is not consistent, and yet in sluiced beaches of a dam the gradation of particle sizes is almost uniform, varying from the largest at the top of the slope to the smallest near or in the pool. If the fineness gradation lines in Fig. 6 are based on Hardy Dam, or similar, materials, Fig. 5 (curves for Hardy Dam) fails to show such a range of variation in the particle sizes of the borrow-pit deposits that would satisfy the conditions of Fig. 6.

EDWARD M. BURD,[26] M. AM. SOC. C. E. (by letter).—One of the discussers, Mr. Sherman, criticizes the writer's generalization concerning a "1 on 5 slope from head-water to tail-water," as being inconclusive without more exactly defined conditions. Perhaps that generalization is inconclusive if it is not read with the presumed limitations explained in the text immediately preceding that reference. Evidently, Mr. Sherman desires, in common with every one, a more exact determination of each problem of percolation, which then could be solved much more precisely by known laws. As was brought out in the paper, however, glacial deposits are too heterogeneous to permit of such treatment, at least within economic limitations. Consequently, rule-of-thumb, but safe, generalizations for the existing materials were suggested, and it is hoped will be understood with the stated reservations.

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[26] Civ. and Hydr. Engr., Consumers Power Co., Jackson, Mich.

The four cases of hydraulic gradient cited by Mr. Sherman are excellent examples of the range of earth dams in general, supplemented perhaps by the 1 on 10 or the 1 on 12 slopes common in "silt" dams as ordinarily built in India, representing the extreme of flatness. The second example cited is representative of materials and conditions usually encountered in the heavily glaciated area, as economically applied to the problem.

The writer approves and acclaims whole-heartedly the remarks of Mr. Marsh, as evidence of a common-sense understanding of the practical, as well as the theoretical, problem. The possibilities of steel sheet-piling are far from understood; the subject is far from being exhausted. In this connection the recent standardization, eliminating many of the best and useful sections, is somewhat disheartening to those forced to deal with deep sand and gravel driving.

Answering Mr. O'Shaughnessy's suggestion of more details of design and construction of concrete core-walls, in addition to the data in Paragraph 6 under "Design and Settlement of Embankments," the usual thickness might well be given. The top 20 ft is frequently made 12 in. thick, increasing in thickness about 6 in. for each successive 20-ft depth. Thickness, however, seems of secondary importance. During construction a difference in water head on either side of any observed wall shows that leakage occurs at joints, form ties, occasional stone pockets, etc. The generally observed decrease in percolation with lapse of time may well be due to silting and plugging of such minor, but numerous, points of leakage.

As pointed out by Mr. O'Shaughnessy, permissible limits of leakage vary widely with geological structure and economy. In glacial drift it surely would be economically impractical and probably physically impossible to limit leakage to rates cited for the Hetch Hetchy Dam. Based on past experience, each new project is considered on its economic merits, rather than on a basis of safety. Safety is a major consideration, of course, but safety limitations in glacial structure usually far exceed the economic balance. As stated in the paper, one dam similar to Hardy Dam, but without a core-wall, safely sustains an initial leakage approaching 20 cu ft per sec. The glacial problem is essentially economic.

As to hydraulic pumping of embankment materials, here, again, generalizations are unwise. As far as can be determined, no floating equipment has ever been known to build an embankment in the glacial region under discussion. The largest operators of such equipment had every opportunity to bid on the Hardy Dam embankment, but could not compete on a cost basis with mechanical transportation.

It is difficult to understand how Mr. O'Shaughnessy can "confute" the actual measured settlement of the Hardy Dam embankment, by what may have happened at the Priest Dam, probably under wholly different conditions. He forgets—and the error is quite common—that the writer is discussing solely glacial-drift materials and conditions; and perhaps the major difference between washed glacial-drift embankments and the more usual earth dams lies in the considerably smaller settlement of the former, and its relatively quick cessation as soon as the pond is filled. As compared with this condition, the usual earth dams of equal height settle at least three times as much, extending over a term of years. The latter condition probably applies to the Priest Dam.

Mr. Parsons brings up the very active question of steam-hydro-electric operation as regards firm and peak power. On a system roughly "half and half," of which Hardy Dam is a part, it is much better to put steam on the base, and build the "hydros" as daily peak plants; that is, 8 to 10 hr per day, 5½ days per week. While generalizations are proverbially dangerous, it would seem that "hydro," in general, is being used more and more on a peak basis, except during high water when it would be wasted if not used continuously.

The old valley notch which Mr. Parsons describes as under Spier Falls Dam, is most interesting, the condition being a common one in glacial valleys, and the necessary treatment being usually a matter of judgment. In Michigan, such material has of necessity been left in place and used as foundation; apparently, as long as it is covered and loaded, it maintains its structural integrity.

As for "quicksand," the writer would never take issue with Mr. Parsons' long experience in these matters, and recognizes the many variations and gradations in such materials of "quick behavior." From personal experience, the writer can corroborate Mr. Parsons' complete success in settling boiling "quicksand" by adding a layer of crushed stone, actually purchased for concrete aggregate. A layer only a foot thick will completely transform a boiling-spring bed into which a man would quickly disappear, into a reasonably stable foundation which will immediately support loads in the order of 1 ton per sq ft.

Mr. Parsons' cautions about dependability and tightness of concrete face paving are well taken. However, they apply practically unchanged to central core-wall construction also. In either case, good engineering and good construction are essential. The writer here suggests that face paving of concrete might well be water-proofed by some flexible, jointed metal, mopped fabric, or similar membrane treatment; or what would be equally effective, mud directly on the face of the more pervious earth fill.

The writer appreciates sincerely the discussion contributed by Mr. Justin. In this paper, it has been the writer's intention merely to discuss in greater detail one particular condition and variation of earth dam construction, as very ably covered in textbooks and other publications by Mr. Justin, whose comments upon the economics of the situation as regards steam *versus* hydro-electric power surely are instructive and pertinent and, as he states, contrary to considerable general belief and some prejudice. There seems to have been a noticeable improvement in the general operating attitude on this subject within the last few years.

In common with several others, Mr. Justin has corrected a rather too general statement in the paper concerning relation of fineness of embankment material and slope of the line of saturation. For fine materials beyond the range of sands encountered in the glacial area, it is common practice, of course, to build more nearly impervious embankments of much steeper saturation slope. Perhaps the writer should have been more specific in limiting his statements to the relatively porous and quick-draining sands economically obtainable from glacial deposits, and with other conditions as normally encountered in the region under discussion.

As regards impervious up-stream layer or membrane compared with the more conventional core-wall, Mr. Justin's comments are quite apt and are made with a clear understanding of the practical difficulties involved with each form of construction. Years of experience with the core-wall both in design and construction, are leading the writer more and more to favor the up-stream form of water cut-off. While it is true that there have been some failures with the earlier forms of this design, it is likewise a fact that there have been failures with central core-walls and all other types of construction which, of course, must be designed on the basis of experience as well as theory. Engineers of the U. S. Reclamation Service apparently have solved successfully the problem of up-stream cut-off and have corrected the earlier difficulties, which were relatively minor, as no major failures were ever chargeable to these difficulties. As far as concerns the usual fear of subsequent settlement of embankment materials, it was one of the writer's major considerations in preparing this paper to show that after several years of close observations on many level points it was proved beyond any refutation that properly placed and water-laid glacial sand embankment materials were notably free from such continuing settlement, as directly contrasted with the usual form of earth embankment on which an up-stream concrete blanket would obviously be an unsafe and impractical form of construction.

On the other hand, a concrete slab is not necessarily the best solution and for many conditions the writer wholly agrees with Mr. Justin that an up-stream blanket of fine material probably is even better, and usually would be less expensive. In general, it is not the form of the up-stream cut-off that is of so much importance as the contention that it should be placed on the up-stream face rather than through the center of the embankment, all other conditions being equal. As for rapid drawdowns of ponds of the size contemplated by present-day projects, there would seem to be but little occasion for any rapidity of the degree necessary to cause unbalanced upward hydrostatic pressure on any up-stream slab construction, because ordinary operation normally draws ponds very little, and slowly. Any disastrous failure, of course, might draw a pond rapidly, but in that case its effect would be of minor consideration as compared with the event itself. The writer only hopes that the paper and its discussion may awaken more active interest in this pertinent design problem as affecting earth embankments.

Correctly, Mr. Pollock points out that the more conservative percolation factors advocated by Bligh (who, perhaps, deserves credit for first developing this idea thoroughly), have proved to be more conservative than necessary, under the particular circumstances of the paper in question. However, this should not detract from the excellence of Bligh's pioneering and, moreover, the writer's generalization probably is limited to the peculiarities of the glacial area. The writer is far from satisfied with the state of knowledge of this matter, in general, and has offered his paper in the hope of developing more information and keener interest. It would seem as if the importance of progress in earth embankment construction, both as regards safety and economy, would justify more intensive study and experimentation with the more porous materials than has been made to date. The relatively impervious clay core type of embankment has been developed and proved quite thoroughly under the leadership of such experts as A. S. Crane, M. Am. Soc. C. E., for example; but similar development of the more porous materials and perhaps with even greater economic advantage still awaits the time, effort, and financial resources of adequate study. For this reason, the writer would hesitate to put forth any generalization or general application of percolation factors developed for this rather limited condition under discussion, which by no means covers the entire general field.

Mr. Pollock raises the question of bulking of sand when moderately moist; that is, a condition intermediate in moisture content between the wet condition as first placed and the dry condition such as prevails later. If there is any evidence of this peculiarity, it has escaped notice on the many construction jobs involving sand embankments of the kind described in the paper. In the ordinary construction process—that is, with water being fed to the top layers at close intervals in order to wash them to place—the material in the embankment probably never dries out to the extent of reaching the bulking point. After the embankment has been completed, however, a time must come when that portion above the saturation line would pass through the bulking range of moisture content. If any such bulking action takes place at Hardy Dam, it has escaped notice as such. Check levels are still taken (1934) about twice a year along the full length of the crest wall (which is the upper part of the core-wall construction); but to date there has been no further change, either up or down, since the initial settlement described in the paper. Possibly the weight of the structure would prevent any volume increase, except close to the surface where it would pass unnoticed.

In washing the embankment material into place, it has been observed many times that the wash water strikes down through the pervious sand very rapidly. Frequently, a large monitor stream will lose itself in a few hundred feet, much as streams in the Western Plains area will drop out of sight at the more pervious reaches of their stream beds. Even while the washing stream is carrying sand, the material immediately beneath it is so hard that the workmen walk all over it at all times, and within a few minutes after an area of sand has been washed to place, and the water has disappeared from the surface, the settlement is so complete that an automobile can be driven without difficulty over the new fill. Obviously, this method of placement secures minimum volume and maximum compactness for the fill of material and a few measurements made in times past have indicated that the volume for a given mass is appreciably less than that occurring in the glacially washed and placed borrow-pits from which the material is taken in the natural state. Whether or not this condition has any bearing upon the bulking situation is unknown to the writer, but at least no indication of volume increase has ever been indicated by the many levels run over the reference points at all times during and subsequent to construction.

Mr. Pollock questions the interpretation of the settlement observations, and apparently the writer has not made the details sufficiently clear. The core-wall footing is centered over the steel sheet-pile cut-off, but is provided with a slip joint so that it can settle down over the top of the sheet-piling without the latter offering any appreciable supporting resistance. This arrangement is considered necessary as a precaution against the core-wall "hanging up" on the piling and possibly leaving unfilled spaces beneath the footing which might form dangerous water passages. Consequently, the full weight of the core-wall is carried directly on the natural and undisturbed earth formation on which it was built, and the core-wall settlement record, which includes the crest wall record also, represents essentially the subsidence of the undisturbed glacial formation of the valley floor. There might be a minor but negligible compression shortening of the core-wall height.

Referring to Fig. 4, the settlement record noted as "Total Settlement of Crest Wall" represents the results of levels taken at a number of points directly at the top of the main vertical core-wall to about December 1, 1930, by which time the bulk of the embankment was all in place. Then, as the pond was filled and the crest wall was constructed, as indicated, in February, 1931, it was necessary to transfer the reference points at the top of the main vertical section of the core-wall to other reference points at the top of the crest wall construction, which obviously would settle with the embankment upon which it was placed. This curve then, beyond December 1, includes some embankment consolidation, noted and estimated to be about one-half; but the 3 in. of total settlement to December 1, 1930, must represent, to all practical purposes, only the subsidence of the valley floor. The penstock settlement curve of the same figure represents reference points on the top of the base slab of the penstock; hence, except for negligible consolidation of the concrete in this base slab, it must represent also the general subsidence of the valley floor, which under the penstock was $2\frac{5}{8}$ in. as of December 1, 1930. The penstock section was constructed on a fairly thick layer of mudstone which presumably spread out and decreased the intensity of loading, with consequent slight lessening of settlement over that represented by the more highly concentrated loading directly under the footing of the core-wall. At least, this would seem to be the probable explanation of this slight difference.

The loading intensity at the bottom plane of the base slabs of both the penstock and the core-wall was computed to be practically the same. The curve of power-house settlement also represents valley floor subsidence except for negligible consolidation of the heavy reinforced concrete construction. The fourth curve, however, noted as "Embankment Consolidation at Deep Section," represents essentially only the consolidation of the embankment section itself, after it was carried to full height. The embankment fill, of course, consolidated during its entire construction, but there was no practical way of measuring this consolidation during the building process, nor any practical consideration that would make it desirable. Only the consolidation after completion to full height, as shown by the curve, would have a bearing on the safety and wisdom of placing construction directly on the surface of the completed fill, which was one of the main questions raised by this paper.

The writer appreciates Mr. Creager's willingness to compare Soft Maple Dam and Hardy Dam because of the similarity of natural conditions, with a difference in methods of construction. Presumably, both methods have their good points, but the combination and comparison offers much interesting and valuable information. The writer visited Soft Maple Dam several times in order to study the formation on which it was built, and he agrees with Mr. Creager that most of the very considerable seepage at Soft Maple Dam is presumably passing around and under the dam through the peculiarly porous glacial drifts, rather than through the dam structure itself. The writer further agrees that the effectiveness of the Hardy type of core-wall in the Soft Maple Dam would be questionable; and probably the cost of it would scarcely be justified by the decrease in total seepage. It would seem apparent from the paper that under such conditions an up-stream type of cut-off would have advantages over the vertical core-wall and the writer agrees that an up-stream blanket of fine material would be more effective at Soft Maple Dam than a core-wall. He is rather inclined to agree also with this up-stream treatment for the usual glacial formation; but as yet no conclusive large-scale demonstration of this method has been undertaken, although it has been used effectively on a small scale on Hardy Dam, and on earlier construction in Michigan. At the Soft Maple Dam site Mr. Creager was faced with a difficult situation which apparently was well handled and nothing in his discussion of the paper was intended as any criticism whatever.

Mr. Ackerman's comments are valuable and practical. If his classification chart (Fig. 7) and definition were used consistently, it undoubtedly would be an improvement to the nomenclature and understanding of earth embankments. As he points out, much confusion has arisen through lack of specific designation when discussing the subject.

As for the ice-thrust considerations upon the design of the intake tower at Hardy Dam, the writer might add that in the latitude of Northern Michigan, ice thrust can and does become a major factor. The intake tower at Hardy Dam was designed to resist that thrust, which might be encountered and which, in this case, would be absorbed to some extent by the fill around the lower portion of the tower, while the bridge to the top of the tower was designed to carry its portion of the thrust back to the embankment proper. As for the side thrust, the shape of the tower is such as to be well braced against side action.

Mr. Holmes' description of ground-sluicing methods by the Northern Pacific Railway Company parallels in economy and effectiveness similar sluicing in the construction of the earlier dams in Michigan under the direction of Mr. William G. Fargo. The first sites developed were selected, to a considerable extent, with a view to direct sluicing from the adjacent high hills and much of this yardage was placed at costs then prevailing in the order of 7 cents per cu yd. Unfortunately, in later developments, the engineers have not been able to avail themselves of topography that would make such sluicing possible, with the result that it has been necessary to move the fill by other and more expensive methods. Where direct sluicing is possible, however, it is believed that no other method has ever approached it in economy and general suitability.

Regarding settlement, Mr. Holmes offers most interesting data relative to the more recently built Fifteen-Mile Falls Dam on the Connecticut River. It is regrettable that similar comprehensive methods of making subsequent settlement measurements available to the profession have not been more widely adopted in dam construction. It will be a number of years, of course, before the final settlement facts of this relatively new dam will be available for use. The final statement of Mr. Holmes that the total settlement in a well rolled embankment is not great, would seem to be somewhat indefinite and perhaps difficult to prove. Fortunately, he has elsewhere contributed[27] perhaps the best available record of settlement of typical sluiced-center, so-called semi-hydraulic types of embankment, being the records for Somerset and Davis Bridge Dams in New England. The writer begs to repeat the generalities of these data at this point in order to show how the settlement records compare with the experience of the sluiced sand dam at Hardy where the 115-ft maximum section of embankment material consolidated only 1¼ in. after being topped out and has shown no further consolidation whatever for about three years since the pond was filled. As compared with this, the record of the Somerset Dam for a maximum height of 105 ft shows[27] a settlement of almost 1 ft in the first 15 yr after construction and even at that time continuing at the rate of approximately ⅜ in. per yr; although this rate is evidently approaching a point of equilibrium at an age of perhaps 25 yr.

The same record shows that the Davis Bridge embankment at the end of the first 5 yr had settled about 0.6 ft in the section approximately 200 ft deep and the similarity of the curves would seem to indicate about the same expectancy. Thus, there appears to be a fundamental difference between the long-continued characteristic settlement of the more impervious earth-type construction as compared with the immediate and complete settlement of much less total magnitude for the water-laid sand dam. This difference is of general observation, although not so exactly measured at other dams; at least the data have never been made available. However, Mr. O'Shaughnessy calls attention to a settlement of 2½ ft of the up-stream fill of Priest Dam, in California, an earth structure 145 ft high, and a 6-ft settlement of the downstream fill of the same structure. Noticeable and evident settlement of this magnitude has been observed by the writer on another semi-hydraulic sluiced core dam of compressible materials of a height of about 170 ft, which was completed several years ago.

The writer acknowledges the correction suggested by Mr. Holmes in connection with the lowest curve on Fig. 4 which, on further thought, should have followed the dots as undoubtedly representing what actually took place rather than as indicated by the smooth rounding off of the curve to which Mr. Holmes quite properly takes exception.

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[27] *Engineering News-Record,* November 14, 1929.

It is always helpful to the Engineering Profession to have the advantage of comments from geologists, who have a further removed perspective on some of the problems treated in this paper, and who, of course, have the advantage of a better understanding of the formation of the site and adjacent region. Technical geological assistance has been of great value in studying these problems in the complicated glacial formation, and it would seem as if a thorough geological study should and must precede any safe and well-organized program of dam construction. For this reason and because of the general information added to the paper, the comments by Mr. Crosby are thoroughly appreciated.

Several of the questions raised by Mr. Hatch have been answered elsewhere in this closing discussion. The writer disagrees with his first two paragraphs in so far as they apply to high dams on pervious glacial drift; and his fourth paragraph evidently represents his judgment of the action of the type of material encountered at Cobble Mountain Dam near Springfield, Mass. This action is not at all characteristic of water-laid sand.

Mr. Hatch requests a "break-up" of the 30-cent cost per cu yd. It is difficult to sub-divide the actual costs of earth handling into separate items from the daily time sheets, and practically impossible to secure a finer classification of costs than those given in the paper. The over-all earth-placing cost is accurate, however, and as finely sub-classified as need be for the purpose of the paper.

As for the last paragraph of Mr. Hatch's discussion, the writer agrees that "hair-splitting" exactness would require a slight shift of the 100-mm line; but the basic idea demonstrated by Fig. 6 is not affected in any way thereby.

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TRANSACTIONS

Paper No. 1875

IMPROVED TYPE OF FLOW METER FOR HYDRAULIC TURBINES

BY IREAL A. WINTER[1], ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. C. MAXWELL STANLEY, R. C. JOHNSON, E. A. DOW, E. B. STROWGER, AND IREAL A. WINTER.

SYNOPSIS

The operation of the modern hydro-electric power plant for high efficiency necessitates the determination of the actual turbine and generator performance curves. Such curves can now be determined to a high degree of accuracy by rating the turbine water input by either the Allen or the Gibson method. These ratings are usually in the nature of a turbine efficiency test made to determine whether the manufacturer's guaranties have been met, or to check the efficiency of old installations. As the performance curves thus established apply to only one head and one plant-loading condition, its utility as a means of presenting operating data has been considered secondary. This attitude has been justifiable, to some extent, as no generally applicable means have been available for rating permanently, a device that would indicate the turbine discharge independent of the head and load on the plant.

The necessity of using, to the fullest extent, the rate of flow data obtained during the turbine efficiency test, led to the development of the Winter-Kennedy principle for obtaining a permanent plant discharge record. The principle utilized is based on fundamental laws of flow, enabling the prediction of performance before installation. Pressure differences existing in the conventional turbine scroll case serve as the prime movers and, therefore, no loss of pressure head, change in direction of flow, or turbulence, is set up in the water passages of the turbine. The means utilized are flush piezometers located on the inner and outer surfaces of the scroll case.

NOTE.—Published in April, 1933, *Proceedings*.

[1] Denver, Colo.

The writer describes the fundamentals of this new method of determining the turbine discharge, and discusses, at some length, the results obtained on models tested in the laboratory and actual plant installations.

INTRODUCTION

The increased efficiency of present-day hydraulic turbines and the added refinements to power plants representing large financial investments, suggests the need of accurate knowledge of the water input and power output.

When the turbine is driving an electrical generator, the determination of the power output is relatively a simple matter. Electrical metering instruments, capable of a high degree of accuracy, are available for every type of service. The measurement of the rate of flow of large quantities of water in the field, however, is still recognized by hydraulic engineers as one of the most difficult of engineering problems.

Methods of rating water passages have been developed by American engineers, to a point where reliable and consistent results are obtained, covering a wide range of conditions. The methods developed by C. M. Allen, M. Am. Soc. C. E. (salt-velocity method[2]), and N. R. Gibson, M. Am. Soc. C. E. (time-pressure method[3]), have met with wide approval. The momentary rate of flow may be determined by both these methods, leaving the problem of permanent plant recording to be solved successfully.

Early attempts to solve the combined problem of rate of flow and plant recording were made, using the Venturi tube. This type of meter has proved practical for high-head impulse installations where ideal pipe-line conditions are to be had.

When attempts were made to adapt Venturi tubes to plants of relatively low head and short penstock, the expense of the tube and loss in pressure head made them economically unsound. It was discovered that Venturi tube characteristics were subject to considerable variations when departures from ideal conditions were encountered. The meter coefficients were found to vary as much as 10% from the ideal, and square law ratios did not exist.[4]

Another attempt to combine rate of flow and plant recording was made with the pitometer. This instrument proved unsatisfactory because it was subject to plugging by trash or because it was knocked out of position by solids carried in the water stream.

Many successful installations of equipment for obtaining permanent plant records have been made by placing piezometers in converging sections of water passages or on elbows in the penstock. As these devices are dependent upon plant conditions, they do not have general application.

It is significant to note that all recording devices for use with plants of large turbine discharge and low head must be rated in the field by some independent method of water measurement. This is also true of the method developed by Mr. A. M. Kennedy and the writer. The advantages of the improved type of flow meter are that it has general application, and its performance can be predicted with sufficient accuracy to enable the preparation of designs before the plant is constructed.

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[2] *Transactions*, A. S. M. E., Vol. 45, No. 1902, p. 285.

[3] *Loc. cit.*, No. 1903, p. 343.

[4] Serial Report, Publication 278-34, National Electric Light Assoc., pp. 6-7.

SCROLL-CASE DESIGN

An understanding of the basis of designing scroll cases is necessary in considering the operation of the differential pressure taps. There are three general methods of arriving at the progressive areas of the scroll, namely, the accelerating velocity method, in which the area progressively decreases relative to the increment of water discharged into the turbine; the constant velocity method, in which the area decreases directly as the increment of flow into the turbine; and the decreasing velocity method, in which the net area of the scroll case is progressively greater than the increment of flow into the turbine.

The three methods involved are based on the fundamental law of conservation of angular momentum, or the law of the free spiral and circular vortices. Only in the accelerating scroll case of the free spiral vortex design does the center of the turbine and vortex coincide.

For structural considerations, a scroll case is not the path following the stream lines of a vortex. Investigation of the flow within the scroll shows a wide range of stream lines into the turbine speed-ring.[5] The discharge varies from a radial flow on the upper and lower surfaces of the scroll adjacent to the speed-ring, to an angular flow greater than the angle of the speed-ring stay-vane at the center line of the distributor.

Concrete scrolls of rectangular section and plate-steel scrolls of circular section are of similar design. The difference in shape is due to structural rather than to hydraulic requirements. The baffles in plate-steel scrolls are located substantially at a point 90° to the transverse counter line of the turbine. In the case of the concrete scroll, considerable latitude is taken in locating the baffle. Efficient designs have been executed with placement at points ranging from 90° to the transverse center line of the turbine, to a point substantially on the down-stream center line, covering an arc of 270 degrees. In all cases the remaining part of the true scroll passage is designed according to one of the three methods outlined.

NOTATION

The following notation is used throughout the paper:

A = area of scroll case at piezometer section.
C = coefficient of deflection.
H = head: H_e = effective pressure head on turbine; H_{v1} = velocity head corresponding to tangential velocity component, V_1; H_{v2} = velocity head corresponding to V_2; H_t = effective pressure head on turbine at time of test; and H_c = a common pressure head selected for convenience of comparing test data.
P = differential pressure between piezometers: P_t = pressure at test head; P_c = pressure at a common head.

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[5] *Transactions*, A. S. M. E., Vol. 53, No. 13, HYD-53-4, p.32.

Q = discharge: Q_t = discharge through the turbine at time of test; Q_c = discharge at a common pressure head; and Q_n = net expected maximum discharge through the piezometer section.

R = a distance measured from the center of the turbine: R_1, R_2, etc. = distances to piezometers; and R_x = distance to center of gravity of piezometer section.

V = velocity: V_1, V_2, etc. = tangential velocity components at piezometers; and V_x = mean velocity at center of gravity of piezometer section.

g = acceleration due to gravity.

h = head: h_v = any velocity head; and h_f = any friction head.

k = experimental constant.

m = mass of water flowing at any point in the piezometer section.

n = logarithmic exponent.

Flow Meter Design

As previously pointed out, the flow lines within the scroll case do not follow definite directions, but are subject to considerable variation. This combination of flows invites an investigation as to their possible effect upon pressures at various points across the section. The following laws of flow are significant:

(*a*) For radiating currents, the pressure head at any point distant from the center is a function of,

$$R_1 V_1 = R_2 V_2 \qquad (1)$$

(*b*) Likewise, for a revolving mass of water in which the stream lines are concentric circles and the total pressure head for each stream line is the same, the pressure head at any distance is a function of Equation (1).

(*c*) Furthermore, for a revolving mass of water having a radiating flow combined with a circular flow, the pressure head at any distance from the center line of the turbine is a function of Equation (1), for both components of flow, and with positive or negative acceleration.

(*d*) If no change in direction or relative magnitude of flow takes place within the scroll case, the differential pressure, P, existing between any two points is found to be,

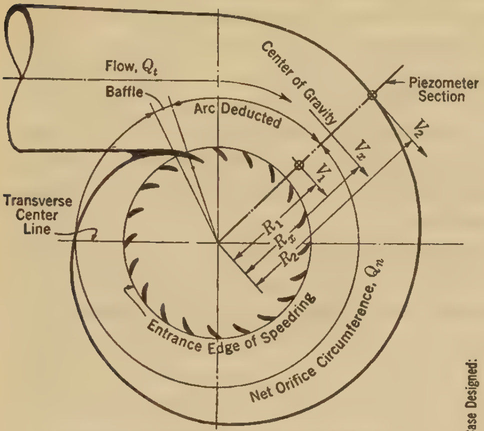
$$P = \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (2)$$

Therefore, the flow, Q_n, through the scroll at the piezometer section is a function of the square root of the differential pressure, P. Laboratory and plant tests covering a wide range of conditions confirm these conclusions.

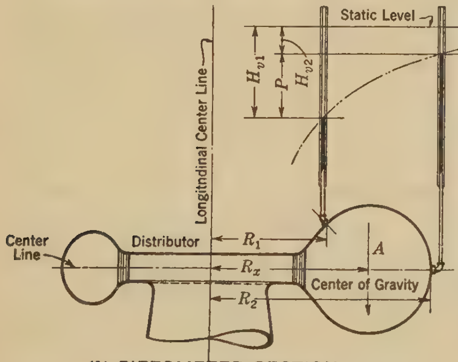
In designing an installation of scroll-case differential-pressure taps, the relation, $R_1 V_1 = R_2 V_2$, is used as a basis. The velocities, V_1 and V_2, are considered as tangential components of the absolute velocity, and the radii, R_1 and R_2, are their distances from the center of the turbine. This assumption is necessary as the absolute direction of the flow is unknown. Therefore, Equation (2) must take the form of,

$$P = C\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad (3)$$

The quantity of water flowing at the section selected for trial, is determined by assuming the entrance edge of the speed-ring stay-vanes as the orifice of discharge from the scroll case (see Fig. 1). The total arc of the speed-ring orifice, through which the total discharge, Q, flows, is determined by deducting the thickness of the scroll baffle on a line with the circumference of the stay-vane tip of the speed-ring. The net quantity, Q_n, flowing past the piezometer section is taken as the percentage of orifice circumference past that section. This establishes the net flow to be used in the calculations. The quantity, Q, when used as a basis of design, is always taken as the maximum expected discharge through the turbine. With the net flow and the area of cross-section of the scroll case known, the mean velocity is,

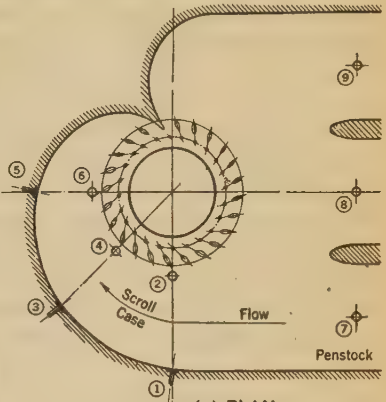

Flow, Q_t
Baffle
Arc Deducted
Center of Gravity
Piezometer Section
V_2
V_x
V_1
Transverse Center Line
R_1
R_x
R_2
Entrance Edge of Speedring
Net Orifice Circumference, Q_n

(a) SCROLL CASE AND SPEEDRING

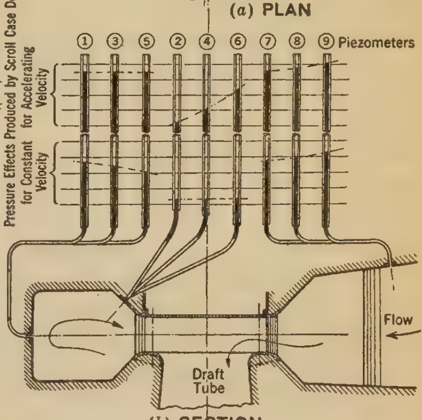

Static Level
H_{v1}
P
H_{v2}
Longitudinal Center Line
Center Line
Distributor
R_1
R_x
R_2
A
Center of Gravity

(b) PIEZOMETER SECTION

FIG. 1.—BASIS FOR DESIGN OF DIFFERENTIAL PRESSURE TAPS


⑤
⑥
④
③
②
①
⑨
⑧
⑦
Scroll Case
Flow
Penstock

(a) PLAN


① ③ ⑤ ② ④ ⑥ ⑦ ⑧ ⑨ Piezometers
Pressure Effects Produced by Scroll Case Designed:
for Accelerating Velocity
for Constant Velocity
Flow
Draft Tube

(b) SECTION

FIG. 2.—INFLUENCE OF SCROLL-CASE DESIGN ON PRESSURE EFFECTS OBTAINED BY DIFFERENTIAL PRESSURE TAPS

$$V_x = \frac{Q_n}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

As shown by the relation, $R_1 V_1 = R_2 V_2$, the increment velocities vary inversely as the radii. Hence, the velocity curve from the entrance of the speed-ring to the outer wall of the scroll case takes the form, $y = cx^n$, with the exponent, n, at unity.

For stability of flow, the revolving mass of water must have a common center of gravity. This common center of gravity is assumed to be the center of gravity of the cross-sectional area of the scroll-case section. Then, by the law of constant moment of angular momentum,

$$R_1\, m\, V_1 = R_2\, m\, V_2 \tag{5}$$

the point of mean velocity, R_x, is about the center of gravity of the cross-sectional area. With the point of mean velocity, R_x, established, the velocity at any point of the section is determined by the relation of its distance (R_1, R_2, etc.) to the center of the turbine.

In order that the differential pressure existing between any two points on the section may be clearly understood, the static and flowing water levels are indicated in Fig. 1. To determine the flowing level at R_1, the velocity head corresponding to V_1, is computed as,

$$H_{V1} = \frac{V_1^2}{2g} \tag{6}$$

Likewise, the velocity head at R_2 is computed as,

$$H_{V2} = \frac{V_2^2}{2g} \tag{7}$$

Therefore, the net differential pressure is,

$$P = H_{V1} - H_{V2} \tag{8}$$

Laboratory Investigations

As the assumption of performance of the proposed differential pressure taps was based on tangential velocity components, it was evident that a coefficient would have to be introduced to obtain the true relation of flow to deflection. The determination of this coefficient was made by use of models in the laboratory.

The first experiments were conducted using sheet-copper models of plate-steel scroll cases. The models were complete with penstock, turbine speed-ring, and fixed guide-vanes. Three types of scrolls were investigated: One with areas corresponding to full acceleration of a free spiral vortex; a second, with acceleration comprising the spiral and circular vortex; and the third, with areas corresponding to negative acceleration. Piezometer sections were established at quarter-points in the cases.

The total rate of flow as determined by pressure readings for the various sections of all three scroll cases was found to be, approximately,

$$Q = k\, P^{0.500} \tag{9}$$

The coefficient, C, in Equation (3), was found to vary with the ratio of height of scroll case at R_x, to height of turbine guide-vanes—the smaller the ratio, the larger the coefficient. The absolute value of C ranged from 0.75 at the entrance to 1.25 at the last quarter of the scroll.

Subsequent tests were made on models of a concrete scroll case in connection with a 16-in. runner, operating under a head of 10 ft. These results proved more interesting as greater deflections were obtained and the influence of the change of angle of turbine gates could be observed.

The characteristics of pressure differences observed in the two types of scrolls are shown in Fig. 2. It is significant to note that for the case with constant velocity areas, the outer, or high-pressure, piezometers (Nos. 1, 3, and 5) record substantially the same elevation. The inner, or low-pressure, piezometers (Nos. 2, 4, and 6), vary as a function of R_x.

For the accelerating velocity case, the characteristics are reversed. The outer piezometers (Nos. 1, 3, and 5), varied substantially as the change in radius of the center of gravity of the scroll section, while the inner piezometers (Nos. 2, 4, and 6), registered equal elevations at various points around the scroll.

These interesting facts suggest that a flow is obtained more nearly approaching the vortex principle than is indicated by the flow lines on the surface of the scroll.

Similar coefficient characteristics as observed in the experiments on the 6-in., plate-steel models were found to exist for the rectangular sections of concrete cases. The absolute value of C is higher for the rectangular section, when based on the ratio of height of the scroll case at the center of gravity, to height of the turbine guide-vanes.

Piezometers Nos. 7, 8, and 9, shown in Fig. 2, are included as interesting information as to the conversion of centrifugal force into pressure head within the scroll proper. Pressures registered by these piezometers are influenced by many outside factors, and a definite relation of flow to deflection does not hold true when compared with each other, or with the pressure units within the scroll.

These differences in pressures over the scroll sections should be more clearly understood by the structural engineer. The eccentric loadings indicated by these tests, introduce bending moments not taken into consideration in the ordinary procedure of designing power houses.

The influence of a change in head on the turbine on the performance of the pressure taps is shown in Fig. 3. The plotting is for readings made on Piezometers Nos. 1 and 2, as shown in Fig. 2. The test points indicated are for five gate-openings, covering a turbine speed ranging from 75 to 115% of the normal designed head (4 to 7 speeds each, over a range from 84 to 110 rpm.). This curve clearly illustrates that no change in characteristics takes place due to a change in angle of the turbine guide-vane, or relative speed of the runner. This is important as there are three factors tending to produce unstable flow conditions at the speed-ring entrance where the low-pressure tap is usually located.

The distribution of velocities into the speed-ring is not symmetrical or uniform about the center line of the distributor. This is due to the lower section of the speed-ring supplying water to a larger area, and the shorter path to the turbine runner. As the guide-vanes are closed, the distribution is more symmetrical because the gate-opening becomes the controlling orifice.

A second influence in the distribution of velocity adjacent to the speed-ring is the relative position of the entrance edge of the guide-vane to the discharge tip of the speed-ring stay-vane. Cases have been known in which the change of flow from one side of the speed-ring vane to the other caused a drop of $2\frac{1}{2}\%$ in over-all turbine efficiency. Any pressure tap within the influence of this velocity change would register unstable characteristics.

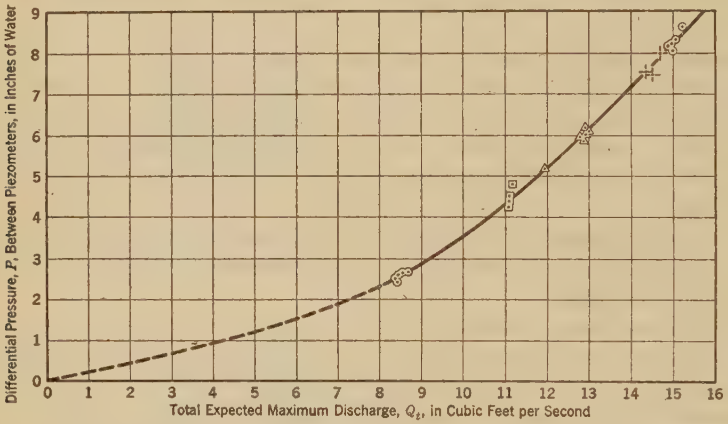


FIG. 3.—DIFFERENTIAL PRESSURES FOR 16-INCH TURBINE MODEL, SHOWING EFFECT OF CHANGE IN GATE-OPENING AND SPEED OF RUNNER

The third factor producing undesirable pressure effects is the constant speed of the runner for all heads. This item is seldom of importance because the head on the turbine rarely ever approaches a value such that a short circuit of flow is likely to take place on the entrance side of the runner.

In all cases tested in the laboratory, the inner piezometers were outside the speed-ring casting. The influence of gate changes and speed of runner could not be found for any combination of conditions.

From the results of laboratory investigations, the following conclusions may be deducted:

First.—For all scroll cases designed according to one of the three methods outlined, the following relation was determined:

$$Q = k\ P^n \qquad (10)$$

Second.—The coefficient, C, was determined as varying from 0.75 to 1.25.

Third.—The coefficient, C, is substantially constant when the laws of similitude are applied. This makes possible the utilization of experimental coefficients for locating pressure taps in any scroll case.

Fourth.—The exponent, n, was found to be substantially 0.500.

Fifth.—The low-pressure taps, when properly located with reference to the speed-ring, are not subject to influence by a change in the angle of the turbine guide-vanes, or to a change in head on the turbine.

Flow Meter Application

Plant Installation.—A typical section through an hydro-electric power plant is shown in Fig. 4. An installation of differential pressure taps is shown, diagrammatically, on the down-stream side of the power house. The differential pressures are related to the static forebay pressure. The notation, $h_f + h_v$, indicates the total friction-head and velocity-head losses incidental to the penstock and other intervening water passages leading to the forebay. This loss in head is shown by the difference in elevation of the water in the high-pressure piezometer line and the static forebay level.

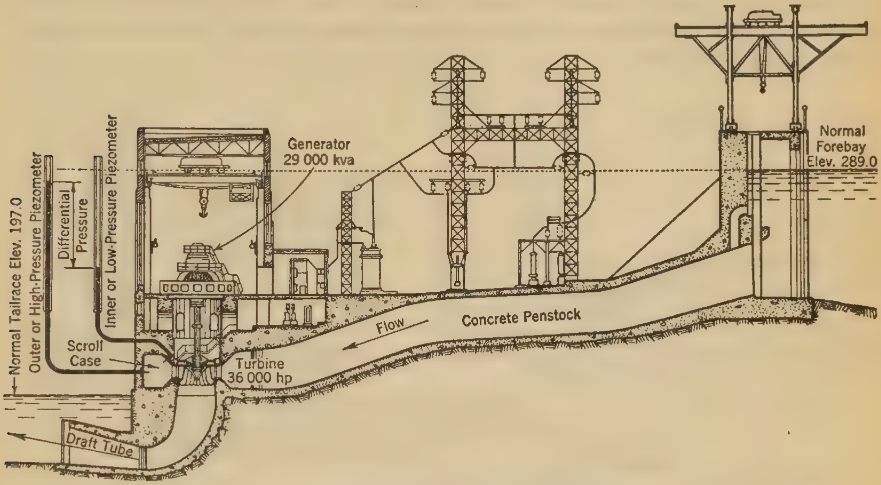

Generator
29 000 kva
Normal
Forebay
Elev. 289.0
Normal Tailrace Elev. 197.0
Outer or High-Pressure Piezometer
Differential
Pressure
Inner or Low-Pressure Piezometer
Scroll
Case
Turbine
36 000 hp
Flow
Concrete Penstock
Draft Tube

Fig. 4.—Transverse Section Through Hydro-Electric Plant, Showing Effect Produced by Scroll-Case Differential Pressure Taps

A typical installation of metering and testing equipment connected to scroll-case differential pressure taps, is shown in Fig. 5. Such a combination gives complete water information for the unit. The mechanical register indicates the rate of flow, in cubic feet per second, draws a chart record showing the change of flow through the turbine, and totalizes the quantity of water passing through the machine, in cubic feet. With these data before him, the operating engineer can determine the efficiency of the unit covering any particular period of time.

The throttling manometer, shown in the illustration, is especially designed for determining the quantity-deflection relation at the time of test. Its several features consist of an engraved scale, graduated in inches and tenths, with the zero of both scales at the bottom. The scales are placed on the center line of the glass tubes, enabling accurate observations on the meniscus of the mercury column. An equalizing valve and suitable air vents provide means of obtaining a clear column of water connecting each leg of the manometer. The throttling valve at the bottom of the U-tube enables the successful dampening of the mercury column without introducing the usual errors inherent with throttling at points in the water line. By use of this manometer, check tests can be made on the performance of the register, and readings on higher or lower pressure taps can be compared with those on the standard taps in case of an emergency.

In Fig. 5, there are three low-pressure taps designated as Piezometers Nos. 1, 2, and 3. Tap No. 1 is a special connection to obtain a large differential pressure. The performance of this tap is influenced by a secondary factor due to the centrifugal force, in a vertical plane, of the water flowing around the curvature of the speed-ring casting. This secondary force is of prime importance as it can reach a value at high flows for which the quantity-deflection ratio departs from the square law. This is due to the tendency of the water to leap clear of the casting as the velocity increases and the restraining pressure decreases.

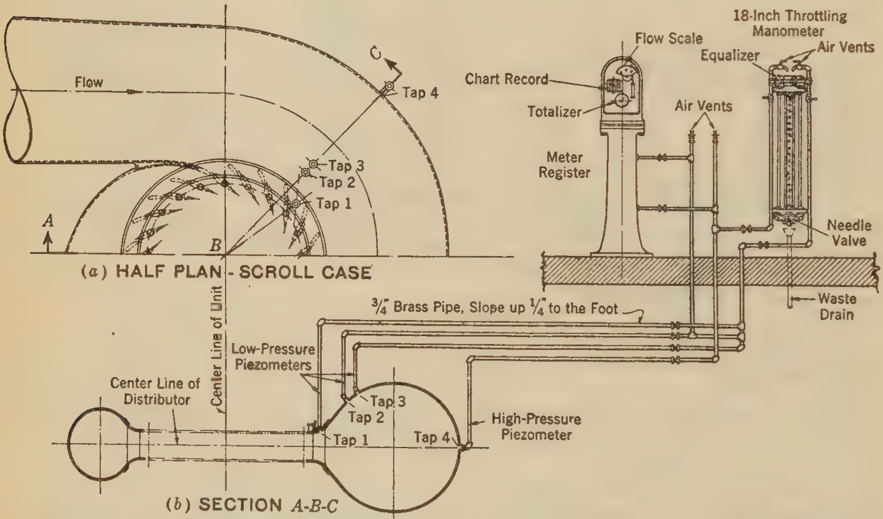

Flow
C
Tap 4
Tap 3
Tap 2
Tap 1
A
B
(a) HALF PLAN - SCROLL CASE
18-Inch Throttling Manometer
Flow Scale
Equalizer
Air Vents
Chart Record
Totalizer
Air Vents
Meter Register
Needle Valve
Waste Drain
3/4" Brass Pipe, Slope up 1/4" to the Foot
Center Line of Unit
Low-Pressure Piezometers
Center Line of Distributor
Tap 3
Tap 2
Tap 1
Tap 4
High-Pressure Piezometer
(b) SECTION A-B-C

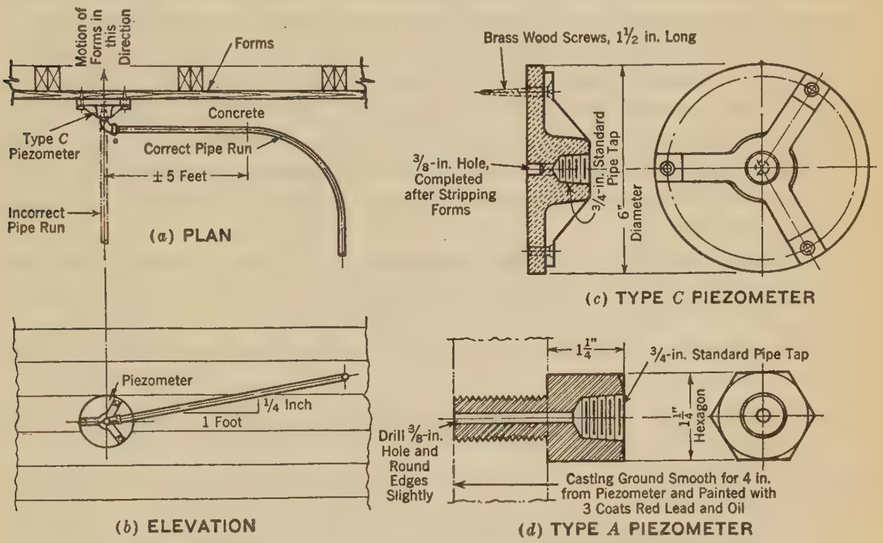
FIG. 5.—TYPICAL INSTALLATION OF METERING AND TESTING EQUIPMENT ON PLATE-STEEL, SCROLL-CASE, DIFFERENTIAL PRESSURE TAPS

Pressure Taps Nos. 2 and 3 are designed as standards to meet the range of deflection of commercial meter registers. In all cases there should be two inner taps so designed that a variation of 20% in flow will produce like pressures. This is necessary, due to a possible increase in expected turbine discharge. If only one tap is installed, the capacity of the register might be exceeded, making necessary the purchase of a special instrument.

The type and method of installing the piezometers within the scroll case are of first importance. Two of the most typical designs are shown in Fig. 6. Fig. 6 (*a*) and Fig. 6 (*b*) illustrate the correct method of fastening piezometers to wooden forms and of extending pipes through the concrete. Details of two types of piezometers are shown in Fig. 6 (*c*) and Fig. 6 (*d*). The Type *A* piezometer is for use with the plate-steel scroll case and speed-ring castings. The Type *C* piezometer (for use with a concrete scroll case) is anchored to the forms with three No. 10, flat-head, brass screws, $1\frac{1}{2}$ in. long. After the forms are stripped, the screws are cut off flush with the face of the brass plate. Thus, after the walls have been cleaned, the drilling of the $\frac{3}{8}$-in. hole is completed so as to make the pressure connection to the $\frac{3}{4}$-in. line.

The reason for drilling the piezometer opening only part way in Type *C* is that cement mortar is likely to run into the piezometer line and plug the passage during the process of pouring the concrete. The completion of the hole after the forms have been removed is a simple matter. By providing the anchorage holes for the brass screws, damage to the piezometers when removing the forms is eliminated.

The Type *A* piezometer is made of monel metal for rust resistance and metallic strength. The threaded shank should never be tapered. Invariably, if it is, the nose of the piezometer will not extend to the inside of the scroll, thus creating a most embarrassing situation.


Motion of Forms in this Direction
Forms
Concrete
Type C Piezometer
Correct Pipe Run
± 5 Feet
Incorrect Pipe Run
Piezometer
1/4 Inch
1 Foot
Brass Wood Screws, 1½ in. Long
⅜-in. Hole, Completed after Stripping Forms
3/4-in. Standard Pipe Tap
6" Diameter
1¼"
¾-in. Standard Pipe Tap
1¼" Hexagon
Drill ⅜-in. Hole and Round Edges Slightly
Casting Ground Smooth for 4 in. from Piezometer and Painted with 3 Coats Red Lead and Oil

(*a*) PLAN

(*b*) ELEVATION

(*c*) TYPE *C* PIEZOMETER

(*d*) TYPE *A* PIEZOMETER

Fig. 6.—Standard Piezometers and Their Installation

The method of making the pipe run adjacent to the scroll case is of importance. The reason for providing a 5-ft run, parallel to the surface of the scroll (Fig. 6 (*a*)), is due to the "giving way" of the forms as the concrete is being poured. When the pipe is run straight out, sufficient embedding is often found to hold the piezometer in place, while the forms move out. This leaves the piezometer recessed in the concrete, and a poor installation results. The same care should be exercised in connection with plate-steel cases as the pipe threads might be stripped in the piezometer nut, permitting leakage in the pressure line.

As the differential pressure taps in the scroll case are capable of indicating flow to a high degree of accuracy, the installation of piezometers and pressure lines should be given extreme care. Close attention should be given to tightness against leaks, proper location of valves, and careful alignment of pipe to obtain a positive upward slope to convenient valves for the elimination of air. The piezometers should have ample clearance from any projection or depression within the walls of the scroll case. A smooth surface, capable of being maintained in that condition, should surround the piezometer.

Plant Calibration.—Pressure taps in the scroll case are of considerable value during the turbine efficiency test. They provide a convenient and accurate means of comparing the measured flow through the machine with the power output, the distance traveled by the piston of the servo-motor, and net head on the turbine. Inconsistent points are weighed as to their probable value, or are discarded as being in error.

The instruments used in measuring the differential pressure should be of the highest type available. The observer should be skilled in the reading of the true pressures and in the manipulation of the pressure lines. Static readings, before and after every test, should be included as routine procedure.

In the case of a plant with two or more units of identical design, it is customary to make a measured test on only one unit. The remaining units are then calibrated for discharge, by using the electrical output as a measure of flow, assuming the efficiency to be the same for all units. This is done to best advantage by reducing kilowatts and cubic feet per second to a common head, H_c, the kilowatts being reduced or increased as $H^{1.5}$, and the cubic feet per second as $H^{0.5}$.

To obtain the discharge value for the machine being calibrated, the kilowatt load is reduced to the common head on which the test curve is based, as $\left(\frac{H_c}{H_t}\right)^{\frac{3}{2}}$. The discharge corresponding to that load is then reduced to the calibration head as $\left(\frac{H_c}{H_t}\right)^{\frac{1}{2}}$. This determines the quantity of water flowing during the calibration run, and establishes the quantity-deflection curve for the pressure taps.

If a manometer is used to obtain the deflection reading, the process may be simplified by reducing the observed values on the manometer to the common head, directly as the ratio of the common to the calibration head (H_c to H_t). This can be seen by the following relation: Since Q varies as $\sqrt{H}$ and P varies as Q^2,

$$Q_c = Q_t \left(\frac{H_c}{H_t}\right)^{\frac{1}{2}} \qquad \text{(11)}$$

and,

$$P_c = \frac{H_c}{H_t} P_t \qquad \text{(12)}$$

It has not been proved that machines of like design and installation have like efficiency. The kilowatt output of a unit is the result (not the cause), of a quantity of water flowing through the turbine. It is important, therefore, that the factors directly influencing the discharge through the runner be taken into consideration.

The principal factors are: (1) The net pressure head on the turbine; (2) the turbine gate-opening, in inches; (3) the diameter of the entrance tips of the runner blades; (4) the discharge area of the runner blades; (5) the speed of the runner; and (6) the degree of air venting into the draft-tube.

The net pressure head on the turbine is determined by net-head piezometers in the lower reaches of the penstock, and by suitable water-level gauges in the tail-race.

The gate-opening is calibrated for every tenth graduation on the governor dial. Three points between each pair of guide-vanes—one at the top, one in the center, and one in the bottom—are calipered for all gates. These values are averaged for the gate-opening curve.

The gate-opening readings are transferred to the outside of the machine by making simultaneous readings on a graduated scale clamped to the piston trunk of the servo-motor that operates the gates. Thus, the gate-opening is translated into the distance that the piston of the servo-motor travels. Consistent readings are obtained by setting the scale at zero on the servo-motor piston with a known oil pressure in the governor system. This pressure should be used at all times and for all machines when setting the scale at zero. The motion of the gates should be in the same direction for making all tests. These precautions are necessary to eliminate the effect of "gate-squeeze" and lost motion in the operating mechanism.

A typical calibration curve for the gate-opening, piston-travel relation is shown in Fig. 7. The governor-dial curve in this diagram is for operating reference and has no significance in connection with testing or calibration.

Factors (3) and (4) relate to the physical dimension of the runner and should be investigated to determine the percentage variation from the tested unit. The discharge area of the runner should be calibrated by triangulation at close points, and the developed sections measured by planimeter so as to determine the outflow areas accurately. The usual method of calibrating the area by taking three or four widths at optional points is not sufficient.

Factors (3) and (5) produce whirls in the water that fill the annular space between the guide-vanes and the runner. These are usually of no importance because the calibration head should not vary greatly from the common test head. The output and discharge relation tends to depart too far from the three-halves and square-root laws when the runner is more than 5% "off speed."

As most hydraulic units are now provided with automatic air vents in the turbine cover-plates, care should be taken that these vents are closed and the automatic mechanism is removed when calibrating the flow meter.

The advantages of making accurate readings of the piston travel to be used as a parameter for plotting the turbine discharge is shown in the flow-travel curve in Fig. 7. The original calibration of the differential pressure taps was made by using the time-pressure method of water measurement.

The subsequent test points were made by manometer readings, and these readings were converted to cubic feet per scond by the equation,

$$Q = 1\,448\ (Hg)^{0.495} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which, (Hg) denotes inches of mercury. Table 1 shows the agreement of discharge measurement as determined by the manometer and as read on the rate-of-flow scale of a standard Venturi tube register after three years of service.

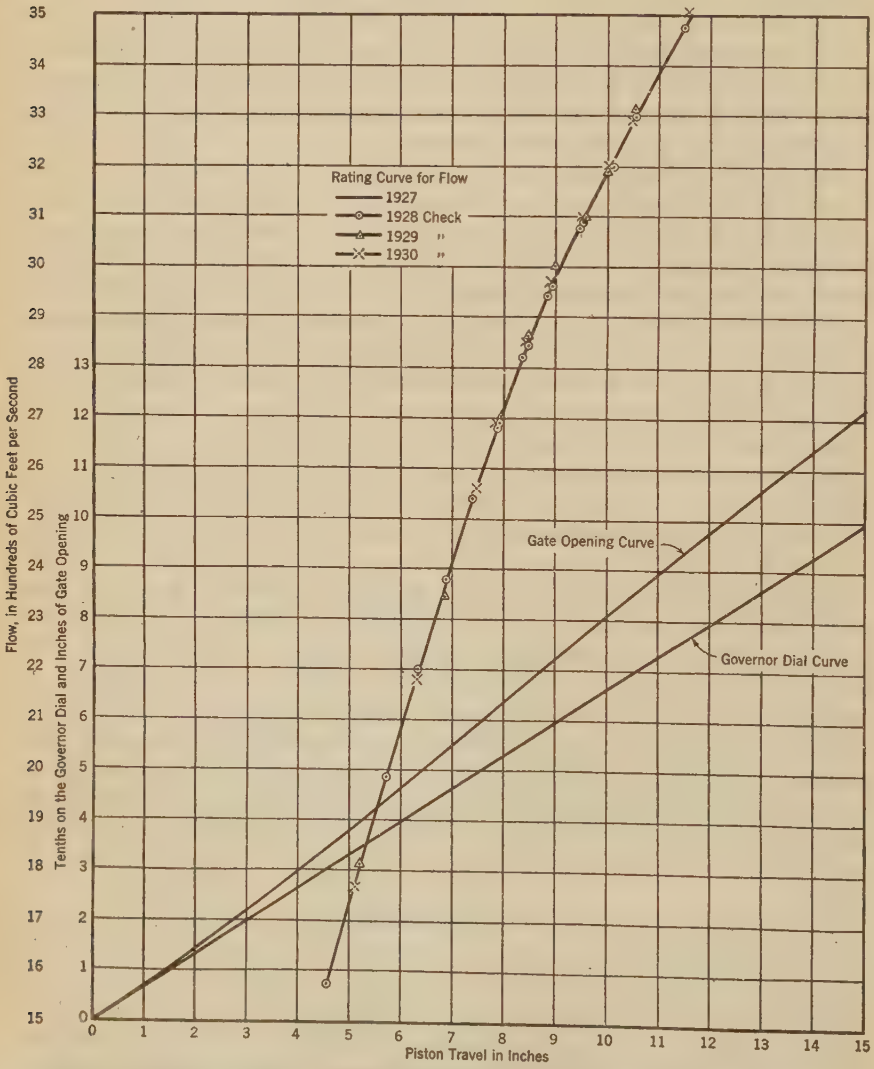


FIG. 7.—RELATION BETWEEN PISTON TRAVEL, AND THE VARIOUS FACTORS SHOWN

TABLE 1.—COMPARISON OF READING BY MANOMETER AND FLOW REGISTER, IN CUBIC FEET PER SECOND

Run	By manometer	By register	Run	By manometer	By register
1	880	878	7	3 100	3 085
2	1 340	1 330	8	3 300	3 300
3	1 770	1 764	9	3 525	3 523
4	2 180	2 172	10	3 210	3 210
5	2 545	2 528	11	2 980	2 968
6	2 850	2 840	12	2 690	2 681

Table 1 lists the actual test values from which the points for the 1930 check test, indicated in Fig. 7, were computed. The installation of equipment is identical to that shown in Fig. 5.

Plant Performance.—The performance of a typical installation of pressure taps in a plate-steel scroll case, is shown in Fig. 8. The piezometer numbers, 1 and 4, 2 and 4, and 3 and 4, refer to the corresponding numbers in Fig. 5. The curves represent test values as determined by using the salt-velocity method of water measurement.

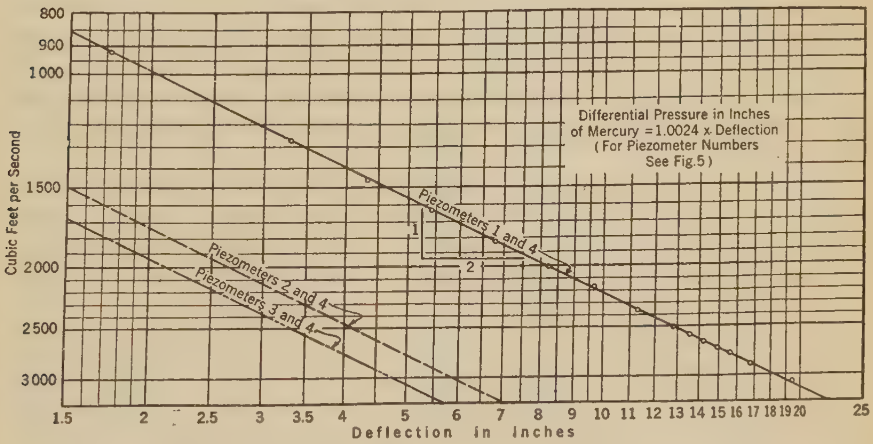


FIG. 8.—PERFORMANCE OF DIFFERENTIAL PRESSURE TAPS IN SCROLL CASE

By inspection, the exponent, n, in Equation (10) is 0.500 for all three combinations of taps. There is no evidence of a change of flow around the speed-ring stay-vane; nor is there a change of the quantity-deflection relation due to the centrifugal force of the water flowing around the short radius of the speed-ring casting. Piezometer Nos. 2 and 4 give deflections in the ideal range for standard Venturi tube registers.

Three similar machines in the same plant were calibrated for turbine discharge by reference to the kilowatt-discharge values obtained during the test, as shown in Fig. 8. The following deflections, in inches of mercury, for Taps Nos. 1 and 4 (see Fig. 5) were found, corresponding to a discharge of 3 000 cu ft per sec:

Turbine Unit No.	Deflection, in inches of mercury.	Percentage variation from the average.
1	21.3	+ 10
2	18.2	— 6
3	19.3	— 1
4	19.0	— 2

The percentages of variation of the four units was found by averaging the deflections. These values are greater than those found for piezometers corresponding to Taps Nos. 2 and 4, and 3 and 4. The predicted and observed deflection for the more conservative designs seldom differ more than 5 per cent.

The reasons for a difference in the pressures found in similar installations are as follows, in order of their importance:

First.—The pressure taps are designed assuming that the water is distributed equally around the unit. This condition does not exist.

Second.—The efficiency is assumed to be the same for like units. Actual data on plant tests, confirming this point, are lacking.

Third.—The cross-sectional area at the piezometer section is assumed to be accurately known. No field measurements are made to determine the point.

Fourth.—The location of the piezometers is not of a precision nature. As long as they are not subject to alteration, no special attention as to exact location is necessary.

Fifth.—When the inner or low-pressure tap is within the influence of the absolute gate-opening, identical quantity-deflection relations are not to be expected. The irregularity of the guide-vane castings and the eccentricity of the guide-vane stem to the face of the casting produce flows of varying

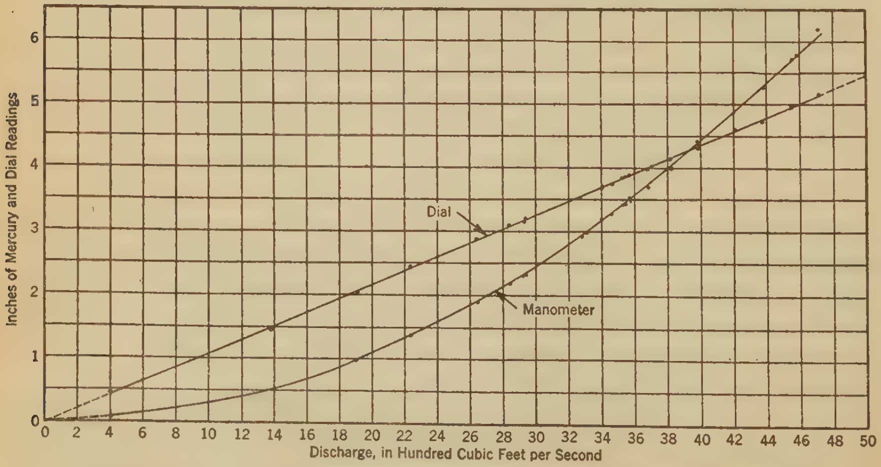


Fig. 9.—Relation of Flow, and Register Indication, and Inches of Mercury Deflection for Differential Pressure Taps

quantities. This fact becomes of first importance in installations using Piezometers Nos. 1 and 4 (Fig. 5).

An interesting exhibit of plant results is shown in Fig. 9. These curves are from plant test data made by the salt-velocity method of water measurement, and are for the plant shown in Fig. 4. The piezometer installation is as shown in Fig. 2, Taps Nos. 1 and 2. The manometer readings, in inches of mercury, compare with the laboratory results, in inches of water, as shown in Fig. 3. The comparative agreement between the laboratory and plant curves is within 2%, the laboratory values being consistently lower.

By plotting the actual test points as given in Table 2, the fine agreement between the measured discharge, manometer, and dummy dial reading of the meter register, can be studied. The readings were obtained on instruments installed as shown in Fig. 5. The meter used is a standard register, designed to operate with a Venturi tube. The flow scale, or dummy dial, is graduated, in inches of the arc traveled by the pointer, and is an arbitrary value used for reference purposes only. By installing the meter register before making the plant test, the values for final calibration are read directly on the scale during the efficiency test. The engraving of a new flow scale is a simple matter, necessitating only the shifting of the instrument zero and re-spacing of the graduations to read in cubic feet per second.

TABLE 2.—PLANT EFFICIENCY TEST DATA

Run	Head, H_t, in feet	Measured discharge, in cubic feet per second	Differential pressure, P. = inches of mercury	Readings, register dummy dial	Horse-power	Inches of piston travel	Inches of gate-opening	Readings on governor dial, in tenths
1....	91.99	1 381	0.51	1.46	10 194	4.18	3.15	3.0
2....	91.97	1 907	0.97	2.04	15 526	5.78	4.45	4.0
3....	91.84	2 639	1.81	2.87	23 933	7.56	5.95	5.0
4....	91.81	2 930	2.33	3.17	26 515	8.28	6.60	5.5
5....	91.79	3 269	2.92	3.53	30 649	9.25	7.45	6.0
6....	91.81	3 527	3.42	3.84	33 233	9.93	8.05	6.5
7....	92.01	3 806	4.00	4.15	35 430	10.93	8.90	7.0
8....	91.82	3 984	4.36	4.42	37 004	11.65	9.50	7.5
9....	91.65	4 207	4.87	4.59	38 312	12.50	10.20	8.0
10....	91.56	4 377	5.26	4.74	39 000	13.25	10.80	8.5
11....	91.33	4 555	5.70	4.96	39 586	14.12	11.50	9.0
12....	91.10	4 719	6.15	5.16	39 902	15.06	12.25	10.0
13....	92.14	2 236	1.36	2.43	19 457	6.62	5.15	4.5
14....	92.09	2 935	2.35	3.20	26 906	8.37	6.65	5.5
15....	92.01	3 291	2.96	3.58	30 986	9.28	7.45	6.0
16....	91.97	3 402	3.21	3.71	32 647	9.62	7.75	6.2
17....	91.98	3 465	3.28	3.76	32 843	9.72	7.85	6.4
18....	91.92	3 544	3.43	3.86	33 773	9.97	8.06	6.5
19....	91.92	3 534	3.44	3.86	33 611	9.97	8.06	6.5
20....	91.88	3 567	3.55	3.89	34 196	10.09	8.15	6.6
21....	92.18	3 681	3.71	3.99	34 657	10.40	8.40	6.7
22....	92.13	3 701	3.77	4.01	34 854	10.53	8.55	6.8
23....	92.04	3 825	4.00	4.15	35 622	10.97	8.91	7.0
24....	92.09	402	0.05	0.46	1 676	1.25	0.90	1.0
25....	92.09	....	0.00	0.00	—228	0.00	0.00	0.0

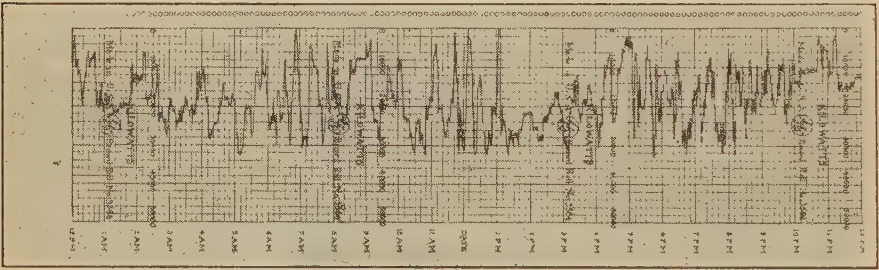
It is significant to note that the quantity-deflection relation for the differential pressure taps, follows those of the Venturi tube sufficiently close to enable the use of standard Venturi tube equipment.

The plotting of the curves in Fig. 9 clearly indicates that a deflection of 6 to 8 in. of mercury is sufficient for all practical purposes. A greater deflection must carry the same error in percentage, and, therefore, can have no advantage over the lower deflection for absolute values. Furthermore, as the rating of the pressure taps is dependent upon the measured flow, a degree of accuracy greater than that of the measuring means is of no value.

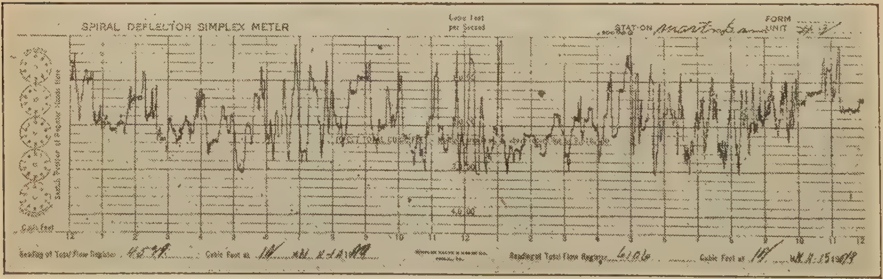
The height of the diagrams obtained in the time-pressure and salt-velocity methods of water measurement do not ordinarily exceed 3 or 4 in.; and a second dimension, time, is introduced in the computation for flow. In the case of the manometer and dummy dial reading, only one dimension is involved. With a deflection of 6 to 8 in. of mercury for the differential taps, the advantages are obvious.

In the case of a calibration test for similar machines where the electrical output is used to determine the turbine discharge, the same relation exists. A high-grade portable watt-meter will ordinarily show a scale deflection of 2 to 3 in. The water-meter register will have a corresponding scale deflection of from 5 to 8 in.

The water-meter register is capable of continuous integration with a prime moving force in the same proportion as the deflection. This method of rating can be used in connection with the measurement of water by current meter and by rotating standards for electrical output, if the engineer believes greater accuracy is obtained by continuous integration.



(*a*) Kilowatt Load Chart



(*b*) Water-Input Rate Chart for Hydraulic Turbine

Fig. 10.—Typical Kilowatt-Load and Water-Input Charts

Fig. 10 (*a*) is a sample record chart showing the flow characteristic obtained for a turbine equipped with a water register connected to differential pressure taps. The corresponding record chart (Fig. 10 (*b*)) of electrical output is included for purposes of comparison.

Plant Operation.—The operating advantages to be had by reference to water-measuring means are not fully realized. The general method in use at present to arrive at the daily operating efficiency is by reference to the kilowatt-hours per hour reading. This value is converted to cubic feet per second by assuming the unit operating at one load, and at constant head. Unless the plant has been on base load for a long time, such methods give results that are not in agreement with the facts.

The following example illustrates the errors possible:

Total kilowatt-hour per hour reading.... = 24 500
Over-all efficiency at 24 500 kw.......... = 89.5%
Apparent operating efficiency.......... = 89.5%

The load was actually carried as follows:

45 min at 29 000 kw.................... = 82.0%
15 min at 11 200 kw.................... = 76.0%

Then, the weighted average efficiency is:

82 × 29 000.......................... = 2 380 000
76 × 11 200.......................... = 850 000

Σ kw......40 200 .... 3 230 000 = Σ products

$$\text{Efficiency} = \frac{3\,230\,000}{40\,200} = 80 \text{ per cent.}$$

Therefore, the error in the efficiency value by using the kilowatt-hour per hour reading is 89.5 — 80.0 = 9.5 per cent. The absolute error in quantity of water flowing through the unit is $\frac{89.5}{80.0} = 1.12$, or 12 per cent. The change in tail-race elevation between hourly readings can frequently account for errors of 2 or 3% in efficiency and flow values.

Concrete proof of the inability of operating engineers to secure the best results by the use of an efficiency curve only is shown by actual plant records. Increases in over-all plant efficiency amounting to 4% in generation have been effected by the use of water registers.

The water power generated in the United States accounts for about 6% of the total generation. As water power can be brought on the line within 1 min to 3 min, with maximum efficiency, the importance of hydro-electric plants as peak-load power can be seen. As the ratio of hydro-electric to steam generation decreases, the necessity of water-recording will increase.

On rivers being developed for power throughout their length, the formation of slack-water pools drowns out the stream-gauging stations. Unless plant-recording devices are provided, the very necessary stream-flow records will cease.

In the case of joint use of a stream by two or more companies, the quantity of water flowing must be known because charges are made for regulated flows. Any method, subject to possible errors as shown to exist when positive recording devices are not provided, might lead to expensive controversies over water rights.

CONCLUSIONS

The following summary of conclusions is offered for discussion, based on the evidence presented in the paper.

(1) Laboratory tests on models of scroll cases indicate that the pressure differences on opposite walls of the scroll case can be used to indicate the rate of flow through the turbine.

(2) These pressures were found to follow definite laws of motion, and a flow relation could be established involving fundamentally sound principles.

(3) Laboratory investigations show that a change in head on the plant will not alter the quantity-deflection relation.

(4) The differential pressures in the scroll case are related to the flow through the turbine with characteristics of the Venturi tube. These characteristics are sufficiently close to enable the use of standard Venturi-tube instruments.

(5) The exponent, n, in Equation (10) ($Q = k\,P^n$) is substantially 0.500. The information being compiled on various plant tests indicates that a true square law relation will agree more uniformly with plant calibrations than the Venturi-tube exponent.

(6) There is no definite information indicating a change in quantity-deflection relation at low flows, as is manifested in the Venturi tube with throat velocities less than 5 ft per sec.

(7) Plant records indicate that the differential pressure taps in the scroll case will give consistent values over a period of years.

(8) The coefficient, C, in Equation (3) was found to cover a wide range, but in agreement with the fundamental laws used as a basis for design.

(9) By application of the laws of similitude, the experimental coefficient, C, can be used to predict the quantity-deflection relation within 5% of the observed value.

(10) Plant recording and operation can be brought to a high degree of efficiency by the use of water-recording devices.

(11) Experience with the scroll-case taps to date, indicate that they are not satisfactory for determining the rate of flow through a turbine without first being calibrated by other means of water measurement.

ACKNOWLEDGMENT

The writer wishes to express his appreciation to the Alabama Power Company, W. S. Barstow and Company, and the Simplex Valve and Meter Company for their co-operation in supplying a large part of the data presented in the paper; and to Mr. Kennedy for his valuable aid in the conception and development of the flow meter.

DISCUSSION

C. MAXWELL STANLEY,[6] JUN. AM. SOC. C. E. (by letter).—The flow meter for hydraulic turbines, which has been developed by Mr. Winter and his associates, and which is described in his paper, fills a decided need in the field of hydro-electric power development. Hydraulic engineers have long desired and searched for a suitable method for obtaining continuous records of discharge of water through turbines for use as a check on the efficiency of operation. Such a method must be simple, positive, accurate, and economical, if it is to have general application.

Numerous methods are available for the measurement of water. These methods include the weir, Venturi meter, and current meter. They also include the salt-velocity method and the Gibson pressure method, which have been widely used for efficiency tests. The list would not be complete without adding the stream-gauging method, which is in general use on rivers. Some of these methods have definite applications, together with definite limitations, as devices for obtaining a continuous record of discharge through a hydraulic turbine or power plant. Obviously, the salt-velocity and Gibson pressure methods, and the use of the current meter, are too complicated for continuous service and must be limited to testing purposes.

The weir is an ideal device for obtaining continuous records of discharge, but it is limited to small quantities, and, even then, it requires a loss of head, which is uneconomical. The Venturi meter may also serve for continuous records where its installation is feasible and economical, but it will be limited generally to relatively small quantities. The stream-gauging method is suitable where a satisfactory control is available and will give good results if used intelligently. Such a station, however, can only measure the total flow and cannot segregate the discharge to various turbines, or to flow over the spillway.

Unfortunately, none of the aforementioned devices is fitted for general application to individual turbines for the purpose of obtaining continuous records of discharge. For such purposes, a device must give a single, unvarying indication for any given discharge and must be susceptible of calibration in terms of true discharge. Such requirements lead into the field of the so-called "index" methods, in which the pressure differential between two points, or the velocity at some one point, is related definitely to the total discharge. Within recent years, considerable attention has been given to the development of such "index" methods for use in obtaining continuous records and as a check against more elaborate test methods.

The use of pressure differentials appears to offer greater possibilities than the use of velocities because pressures are more stable than velocities, and the instrument for measuring pressures—the piezometer—is a more simple and more reliable device than the instruments for measuring velocity—the Pitot tube, or the current meter.

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[6] Engr. (Young & Stanley, Inc.), Muscatine, Iowa.

The Winter-Kennedy method is an ingenious application of the pressure differential method to scroll-case installations. This paper outlines the design and application of such equipment in an interesting manner. The writer believes that the principles of this method may be extended to a wider application to fit cases other than those having closed scroll cases. Whenever there is a flowing water, there will be pressure differentials between various points which will result from friction losses, changes in velocity, or centrifugal action. In all these cases, the pressure differential will probably be a function of the square of the discharge. It is probable that differentials resulting from changes in velocity, or centrifugal action, will be more reliable and consistent than those resulting from friction losses, as the friction losses tend to vary with time

Wherever pressure differentials are of sufficient magnitude to allow satisfactory measurement, and wherever they are a definite function of the discharge, they may be utilized as an index of the discharge; with proper calibration, they may be used to express the total discharge. No doubt, in many instances, these differentials will have to be measured in inches of water rather than in inches of mercury, and equipment will have to be arranged to fit the particular application.

As an example, assume an intake structure to a tunnel with a water velocity of 2 ft per sec immediately behind the trash racks, and a velocity in the tunnel of 8 ft per sec. This change in velocity involves a pressure differential of nearly 1 ft of water which will occur in a relatively short distance. Such a differential is capable of measurement. Other applications might be feasible at bends of penstocks or pipe lines, and at various points on the turbine speed-ring, guide-vanes, or gates of units having an open flume setting. Of course, such installations will require careful investigation and experimentation to obtain reliable results.

R. C. Johnson,[7] Assoc. M. Am. Soc. C. E. (by letter).—The author has presented clearly the mathematical reasoning and experimental evidence leading to very definite conclusions. An understanding by the profession of the operation and installation of this improved type of flow meter should certainly lead to its more general adoption.

In addition to the operating advantages mentioned, such flow meters should offer particular advantages when used in connection with adjustable-blade propeller turbines, where the operating conditions sometimes vary through much wider limits than for ordinary installations. Continuous flow records for the purpose of checking efficiencies would be especially useful in connection with hand-operated adjustable-blade runners. The use of an efficiency curve as the sole guide in the operation of a unit is indeed inadequate. The character of an efficiency curve is by no means constant even under the same apparent conditions of operation. An easy method of checking the efficiency quickly, under any combination of operating conditions, would often lead to the location of sources of waste which would otherwise go unnoticed.

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[7] Assoc. Prof., Civ. Eng., Univ. of South Carolina, Columbia, S. C.

The author points out the eccentric loading condition due to differences in pressures over the scroll-section and the resulting bending moments produced by it. Expressing the pressure difference by Equation (3), with C having a range of values from about 0.75 to 1.25, it can be seen that the pressure difference between any two points in the scroll case could scarcely be sufficient to cause undue annoyance in the structural design, yet stresses produced by such pressure should certainly be taken into consideration. Fig. 8 shows the differential pressure between Piezometers Nos. 1 and 4 (Fig. 5) to be about 22 in. of mercury (approximately 25 ft of water), for maximum discharge conditions. While moments produced by such eccentric loads could not be enormous, a knowledge of their magnitude and distribution throughout the entire scroll-section, based on experimental investigations, would remove considerable guesswork now necessarily used by the structural designer, resulting in a more economical use of materials.

E. A. Dow,[8] M. Am. Soc. C. E. (by letter).—An interesting presentation of a most useful method of metering the discharge of large hydraulic turbines is presented in this paper. Undoubtedly, it will be applied with increasing frequency in future hydro-electric practice.

The turbines of the Comerford Plant of the New England Power Association System (15-Mile Falls development, near Littleton, N. H.), are similar to that from which the data shown in Fig. 8 were obtained, and the results of the calibration test at this plant may be of interest for comparative purposes.

The Comerford turbines are equipped with a number of piezometers to provide a wide variety of differentials for use as mutual checking systems in connection with the measurement of turbine discharge by the salt-velocity method. Among these were four piezometers installed in accordance with designs furnished by Mr. Winter and corresponding to Piezometers Nos. 1, 2, 3, and 4, shown in Fig. 5. In Fig. 11, the performance of these piezometers is presented in logarithmic form similar to Fig. 8, except that the differentials are expressed in feet of water. The data are for the piezometers corresponding to Piezometers Nos. 2 and 4 of Figs. 5 and 8, these being the ones observed during the salt-velocity test.

The solid line ($Q = 1\ 355 H^{0.490}$), represents the calibration formula derived from twenty test runs. This was arrived at by least squares to eliminate personal bias, and assumes equal probability of errors in determining the differentials and the discharges. The dotted line ($Q = 1\ 343 H^{0.500}$), represents a square law calibration, with the coefficient (0.500), assumed to make the average of the ratios of computed discharge to salt-velocity discharge equal to unity.

It will be noted that the two calibrations intersect at about 2 100 cu ft per sec, or 65% of capacity; and that the square law discharge is 101% of the logarithmic discharge at 100% of capacity, and 96% at 10% of capacity. It is obvious that the square law coefficient may be slightly modified to cause the line to pass through some preferred point, sacrificing the average agree-

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[8] Designing Engr., New England Power Eng. & Service Corporation, Boston, Mass.

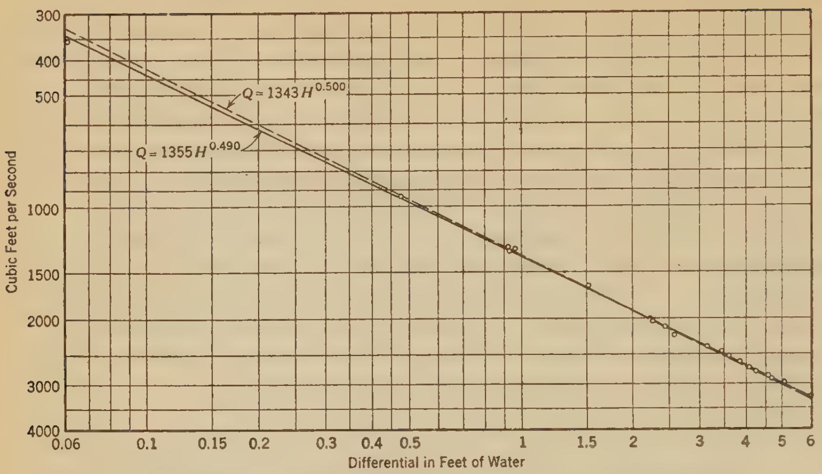

FIG. 11.—PERFORMANCE OF PIEZOMETERS IN TURBINES OF COMERFORD PLANT.

ment with the salt velocity to some extent. The calibration thus reduces to a choice of assumptions, either that the square law is inviolate, or that, on the average, the measured discharges are correct.

E. B. STROWGER,[9] ESQ. (by letter).—The flow meter described by Mr. Winter seems to be in fairly wide use in the case of relatively recent hydraulic turbine installations. This has been brought to the writer's attention by the increasing number of turbine tests made by the Gibson method where a rating of casing piezometers is specified as a part of the work. Of the turbines tested under the direction of Mr. Gibson during the four years (1929–1933), nearly one-half have had casing piezometers installed. The inference that may be made from this trend is that some, or perhaps all, of the advantages pointed out by Mr. Winter for the installation of these piezometers, are becoming generally recognized.

In thirty-three recent tests by the Gibson method, the exponent, n, in the author's Equation (10), was found to vary from 0.484 to 0.544, the average value being 0.517. These data seem to confirm the author's Conclusion (5) which states that the value of the exponent, n, is substantially 0.50 (see, also, Equation (9)). The relation between the computed value of P and the test value is expressed by the coefficient, C, of the author's Equation (3). In the tests cited this coefficient varied from 0.65 to 1.27, the average value being 0.99. These values roughly check Mr. Winter's experiments involving the range in C from 0.75 to 1.25. This relatively wide spread in the value of C is probably accounted for mainly by (1) the assumption that the velocity at each of the taps can be calculated by use of the vortex law; and by (2) neglecting the effect of the centrifugal force of the water as it flows around the case. This wide variation in C makes it impossible to use the casing piezometers as a primary method of water measurement but, as pointed out by the author, the piezometers should be rated by an approved method, such as the Allen or Gibson method, and thereafter may be used for measuring purposes.

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[9] With Buffalo, Niagara & Eastern Power Corp., Buffalo, N. Y.

In designing the piezometer and metering equipment, it is important, of course, to know the relative amount of mercury deflection to expect for the full-gate discharge. Some allowance must be made for the possible error involved in the calculation and, as a practical matter, this may be taken care of by adding two more low-pressure taps, one on either side of the main one, as indicated by the author.

IREAL A. WINTER,[10] ASSOC. M. AM. SOC. C. E. (by letter).—The importance of using pressure differences as a means of determining the flow of water in hydraulic channels has been pointed out by Mr. Stanley. He thus states a fundamental rule that makes possible the correct approach to the problem of measuring, continuously, water in motion.

The structural significance of varying pressures across the scroll case, as noted by Professor Johnson, are not important in connection with rectangular concrete sections, where the structure is usually designed as a cantilever with no external loads carried by the turbine speed-ring. The water pressure inside the scroll tends to relieve the stress and increase the equilibrium of the structure. With plate-steel scroll cases, under relatively high heads, the eccentric loadings do introduce secondary stresses of appreciable magnitude.[11]

This loading may best be illustrated by considering the scroll case empty and superimposing the unbalanced water pressure, due to the difference in velocities across the scroll, external to the casing. Under this loading the casing will assume a shape that will introduce considerable stress in the extreme fiber of the plate at the junction of the plate and speed-ring. The secondary stress, added to the ring stress due to internal loading, is likely to reach a value several times that used in the assumptions for design purposes. As pointed out by Professor Johnson, an understanding of these loadings will result in a better design of structure with perhaps an actual, instead of an imaginary, factor of safety.

The exponent, n, in Equation (10) is of interest in connection with all prime movers that operate registers. As a matter of practical application, the indication of a register must follow within certain limits of deflection. Fig. 9 illustrates the agreement of a Venturi tube register with the exponential characteristic of the scroll-case meter. Figs. 8 and 11 differ in curve slope from each other and from the Venturi tube. It is always important that the register be read at the time of calibration as the lower values of differential pressure are more easily read on the register dial than can be determined by the manometer. With manometer readings being converted into flow from calibration equations or curves, the slope of the quantity-deflection curve is of no moment.

The interesting comparison made by Mr. Dow in Fig. 11, between the basic and actual exponent determined by test, does not indicate that either the test or the differential pressure is in error. It is more likely that both are correct. As a matter of further interest regarding the exponent, n, of the quantity-deflection curve, Mr. Strowger reports values determined by tests ranging from 0.484 to 0.544. These values are significant when considered with an understanding of the relation of the low-pressure and high-pressure taps to the mean velocity at the measuring section.

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[10] Denver, Colo.

[11] *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 101.

Investigation as to the slopes of the quantity-deflection curves for various prime movers where the velocities of the threads of flow vary considerably, shows the same trend of exponential variation. A Pitot tube, when placed in one thread of flow, and calibrated to indicate the quantity flowing in a relatively larger body of water, may have a slope character of wide variance from the square law. In the case of large Venturi tubes or converging sections in penstocks, as referred to in the paper, the slope exponent is not at all consistent with the square law ratio. This apparent inconsistency is a matter for further research.

It may be stated that any pressure or impact orifice, subject to the action of a single thread of a relatively large stream of flowing water will tend to function according to Bernoulli's theorem of the relation of velocity and pressure heads for that particular thread of flow. This statement is at odds with experience in connection with small conduits in which a piezometer ring is said to give the net mean pressure head for the piezometer section. A velocity traverse of the section will show a wide range in velocities without a corresponding difference in pressure.

This apparent phenomenon has been the source of amazement to hydraulic engineers since the relation was determined. It is not inconceivable that Mr. Dow and Mr. Strowger should consider inviolate the theory that a single pressure tap should integrate the entire stream of flow.

As a matter of exact science, according to the fundamental laws of flow, the pressures and velocities should be integrated for the entire flow stream. This process is approached by traversing a measuring section with a Pitot tube of both impact and pressure orifices. The exponential function of the Pitot tube will be found to approximate the square law without measurable differences.

The pressure taps in a scroll-case flow meter intersects threads of flow of widely different velocities in the same plane of pressure. One thread may be leading or lagging in relation to the other, and hence the exponent, n, in Equation (10) may vary between the limits as found without departing from a true relation of velocity to pressure heads.

In presenting the fundamentals of the flow-meter design, the complexity of the relation of flow to vortex pressures has been purposely omitted. When the inherent limitations are considered, of a turbine installation for determining (independently of other means) absolute discharge, it is obvious that a direct approach with easily determined functions, should be used as a matter of interest and utility.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1876

THREE-SPAN CONTINUOUS-TRUSS RAILROAD BRIDGE, CINCINNATI, OHIO

By WILSON T. BALLARD[1], M. Am. Soc. C. E.

WITH DISCUSSION BY MESSRS. J. E. WILLOUGHBY, AND WILSON T. BALLARD.

SYNOPSIS

A description of the newest bridge of the Chesapeake and Ohio Railway Company across the Ohio River at Cincinnati, Ohio, is given in this paper. The recorded history of this important crossing begins with a paper[2] by William H. Burr, M. Am. Soc. C. E., entitled "The River Spans of the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company." The purpose of the present paper is to provide the next chapter of this history. The bridge built from 1927 to 1929 to replace the old one (which was completed in 1888, and strengthened during 1916) is a double-track steel structure, continuous over three spans.

Features of the new bridge that are of special interest are: (1) The trusses (which are the longest of their kind constructed to date); (2) the location of a fixed bearing at the end of the bridge, necessitating an exceedingly heavy anchorage pier of unusual design; (3) a river pier comprising an old, stone masonry part on a timber caisson joined with a new concrete part on a concrete caisson; (4) erection without inconvenience to railroad traffic on the old bridge, or to navigation of the river; and (5) the use of silicon alloy steel for the purpose of reducing dead load.

THE OLD BRIDGE

The old double-track bridge of the Covington and Cincinnati Elevated Railroad and Transfer and Bridge Company, constructed in 1886 to 1888, provided the Chesapeake and Ohio Railway Company and the Louisville and

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NOTE.—Presented at the meeting of the Construction Division, New York, N. Y., January 22, 1931.

[1] Vice-Pres., The J. E. Greiner Co., Baltimore, Md.

[2] *Transactions*, Am. Soc. C. E., Vol. XXIII (August, 1890) p. 47.

Nashville Railroad Company an entrance to Cincinnati, Ohio, from the east and south. The railroad tracks were between the trusses. A 10.5-ft wide, vehicular roadway and a sidewalk, 5.75 ft wide, were supported on brackets outside the trusses on each side of the bridge. The trusses were designed for a live load on each track consisting of two Consolidation engines weighing 119 000 lb each (exclusive of tender), followed by a uniform train load of 2 500 lb per ft; and a highway live load on each wagon way and sidewalk, of 60 lb per sq ft, which is equivalent to 960 lb per lin ft per truss.[8]

The floor system and primary truss members, in addition to the foregoing loads, were designed for a load of 15 tons uniformly distributed in 10 ft, followed, and preceded by, a uniform load of 80 lb per sq ft, on each vehicular roadway and sidewalk. This loading was applied alternately on each side of the bridge. The floor system and lateral and transverse systems of bracing were wrought iron. All main truss members were steel.

An inspection of the bridge made during June, 1916, disclosed the fact that the bridge was carrying locomotives approximately 55% heavier, and train loads, 100% heavier, than those for which the structure was designed. All operation of these loads over it was limited to 15 miles per hour. Subsequently, the bridge was strengthened in some of its members, and it continued to carry the foregoing loads safely for a period of thirteen years until in March 1929, the new bridge was put into service.

The first bridge was maintained in service as a railroad bridge for a period of about forty-one years. It has now been converted into a highway bridge, which should continue in service indefinitely, or for as long a period as its owners take proper care of it.

Description of the Project

During 1926 the Chesapeake and Ohio Railway Company began extensive improvements in Covington, Ky., and in Cincinnati. This work included: (*a*) Elimination of several grade crossings in Covington, necessitating a material rise in grade on the approach to the Ohio River Bridge; (*b*) construction of new passenger platforms and train-sheds in Covington; (*c*) revision of viaducts and trackage in Cincinnati; and (*d*) replacement of the old bridge by a new double-track bridge designed for Cooper's E-70 live load. The construction of the new bridge across the Ohio River was justifiably included in the program of improvements, because it would permit the Railway Company to operate its heaviest equipment between Covington and Cincinnati, with resulting improved efficiency and economy in handling cross-river traffic, instead of being limited, in hauling trains across the river, to the use of power approximately equivalent to Cooper's E-40 loading.

General Conditions and Selection of Design

The old bridge (see Fig. 1) consists of three, simple, through-truss spans, resting on stone masonry piers. The river piers rest on cribs of 12 by 12-in. pine timbers with interstices filled with concrete. These cribs, which are approximately 35 ft high, were constructed on caissons built of pine timber

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[8] *Transactions*, Am. Soc. C. E., Vol. XXIII (August, 1890), p. 62.

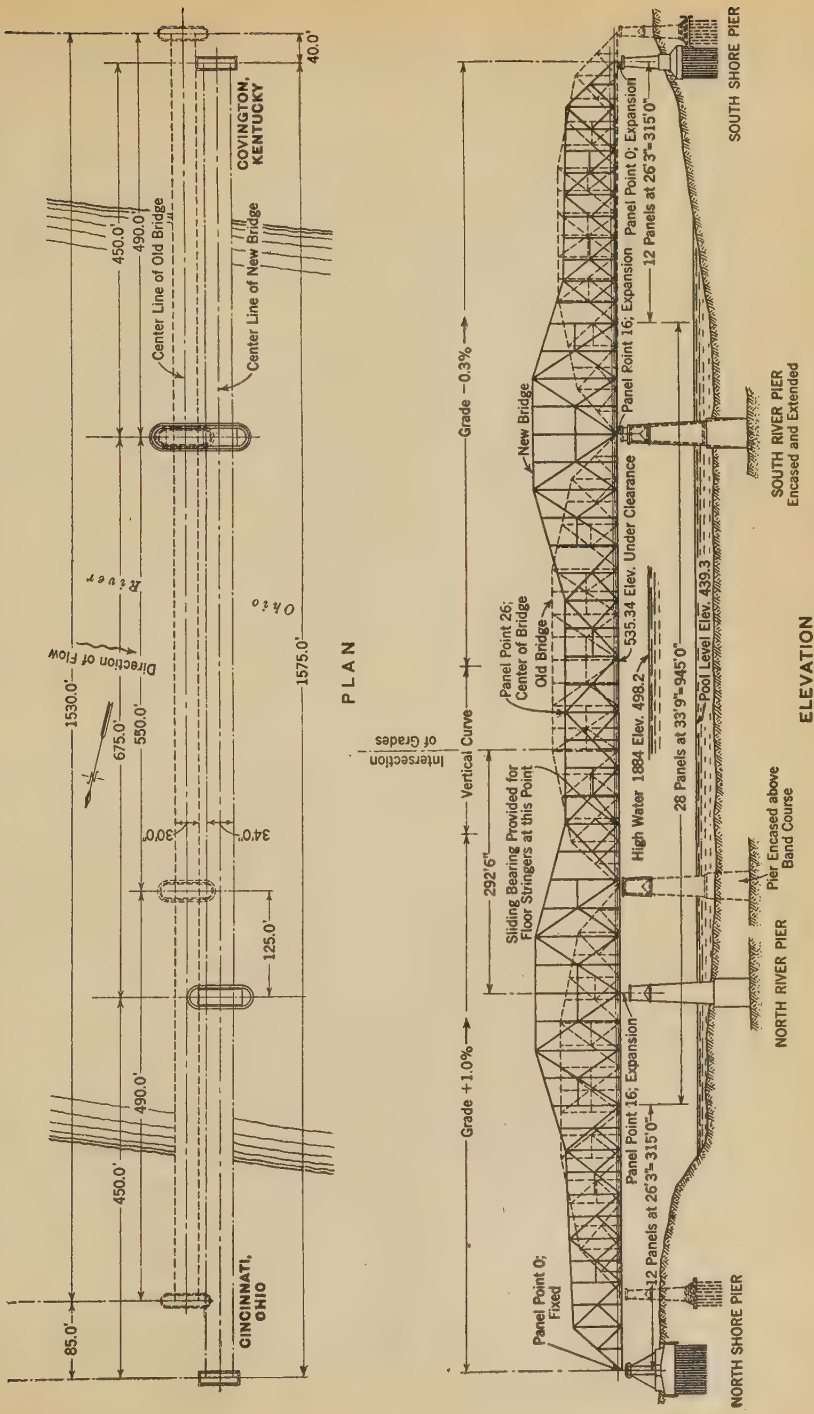

COVINGTON, KENTUCKY
40.0'
450.0'
490.0'
Center Line of Old Bridge
Center Line of New Bridge
Ohio River
Direction of Flow
1530.0'
675.0'
550.0'
1575.0'
30.0'
34.0'
125.0'
490.0'
450.0'
85.0'
CINCINNATI, OHIO
PLAN
Grade -0.3%
Vertical Curve
Intersection of Grades
292'6"
Grade +1.0%
New Bridge
Old Bridge
Panel Point 26; Center of Bridge
Sliding Bearing Provided for Floor Stringers at this Point
Panel Point 0; Fixed
Panel Point 0; Expansion
Panel Point 16; Expansion
12 Panels at 26'3"=315'0"
28 Panels at 33'9"=945'0"
535.34 Elev. Under Clearance
High Water 1884 Elev. 498.2
Pool Level Elev. 439.3
SOUTH SHORE PIER
SOUTH RIVER PIER Encased and Extended
Pier Encased above Band Course
NORTH RIVER PIER
NORTH SHORE PIER
ELEVATION

Fig. 1.—Plan and Elevation of Bridge

with oak cutting-edges and working chambers filled with concrete. Explorations of the foundations of the old piers revealed the fact that the timber of which they were built was sound and in good condition after more than forty years of continuous submersion. According to record, the old piers were founded on rock from 53 to 54 ft below low water. The outer shell of masonry of the two river piers below the band course was found to be in good condition except for the need of pointing. Holes drilled into the bodies of the piers revealed the fact that there were voids in the masonry backing behind the face masonry. The up-stream ends of the top stems of both piers above the band course are split. Prior to completion of the new bridge they were held together by pairs of steel clamps.

At the bridge site the river has a maximum variation of approximately 70 ft between high and low-water stages. Core borings made along the line of the bridge indicated that after passing through approximately 40 ft of over-burden, consisting mostly of sand and gravel, a satisfactory and unyielding foundation of alternate thin laminations of hard shale and sandstone would be found at approximately 66 ft below pool elevation, or about 54 ft below low water. This corresponds with the foundation depth of the original piers. Borings at the sites of both new shore piers indicated conditions that would require pile foundations. The north shore pier is situated on an old fill of cinders and miscellaneous material underlaid by clay, sand, and gravel.

The train movement over the old bridge was almost continuous and it was desirable, if not absolutely necessary, that this traffic should be maintained without interruption during the construction of the new bridge.

Several different plans were considered to insure continuous operation. Some of them involved extending the existing piers and building the new bridge on these extensions, as close as possible to the old bridge, which was to remain and to be converted into a highway bridge. Other plans, similar as to the piers, involved ultimate removal of the old bridge spans to make way for widening the railroad bridge to four tracks, either with, or without, vehicular driveways and pedestrian sidewalks on brackets outside the trusses. Still other plans contemplated span lengths greater than those in the old bridge, and, ultimately, the complete removal of the old structure (including all piers), except any pier that might be incorporated in the new work.

None of the plans contemplating the extension of both existing river piers could be adopted because the War Department refused permission to construct the bridge with the channel span the same length as that of the existing bridge. It was argued that in the future it might be desirable to increase the channel opening by removing or altering the old bridge. Various plans with channel spans longer than that of the old bridge (and requiring the construction of two new channel piers) proved either too costly or involved interruption to railroad traffic during their construction. Finally, the plan in Fig. 1 was adopted.

The availability of unyielding foundations, the span lengths required, and the necessity for providing a bridge of maximum rigidity afforded an excellent opportunity for the economic use of a continuous truss bridge to rest on four piers. This was the type adopted.

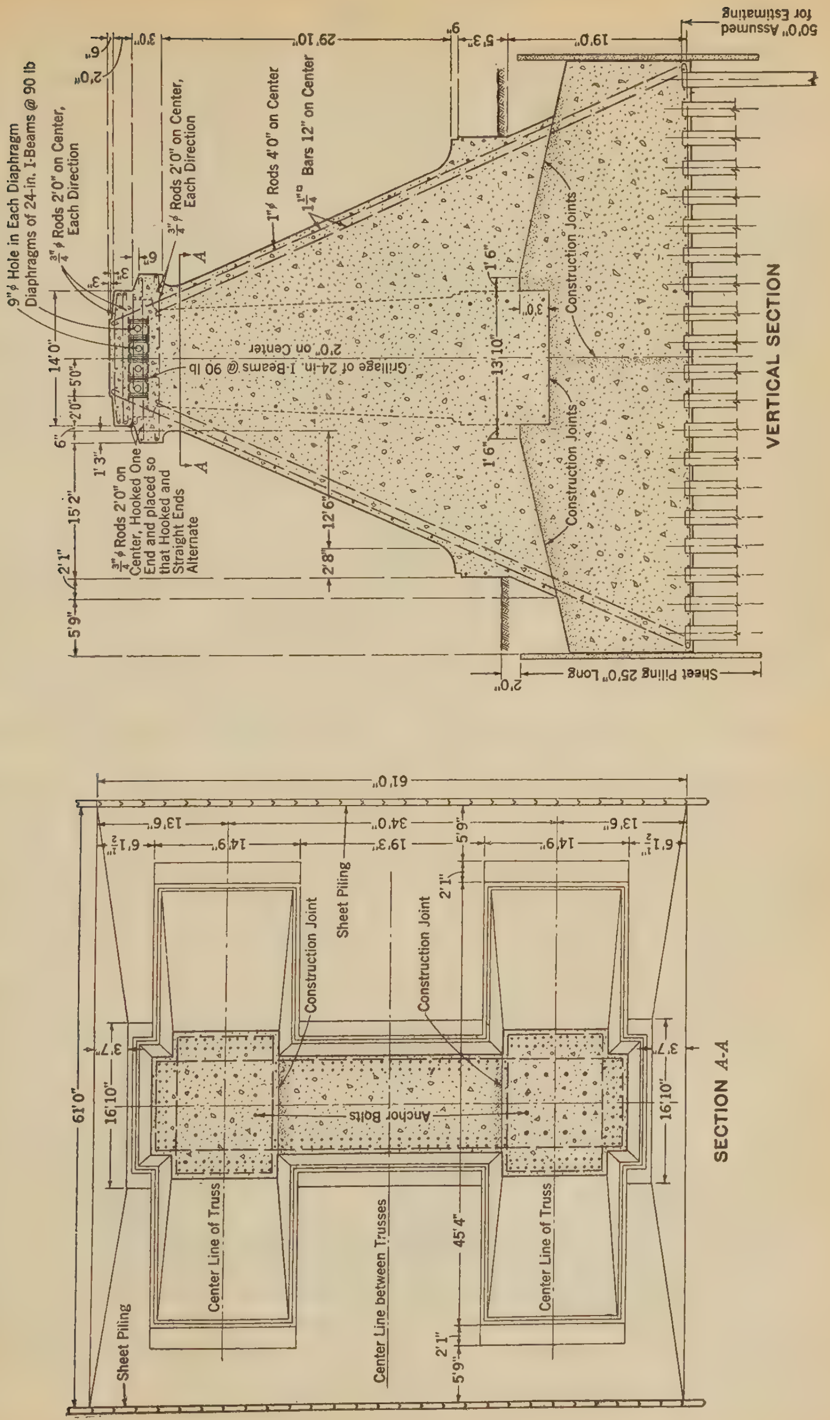

VERTICAL SECTION
9"ϕ Hole in Each Diaphragm
Diaphragms of 24-in. I-Beams @ 90 lb
3/4" ϕ Rods 2'0" on Center, Each Direction
3/4" ϕ Rods 2'0" on Center, Each Direction
1"ϕ Rods 4'0" on Center
1 1/4"□ Bars 12" on Center
Grillage of 24-in. I-Beams @ 90 lb
2'0" on Center
3/4" ϕ Rods 2'0" on Center, Hooked One End and placed so that Hooked and Straight Ends Alternate
Construction Joints
Construction Joints
Sheet Piling 25'0" Long
50'0" Assumed for Estimating
A
A
14'0"
5'0"
2'0"
6"
1'3"
15'2"
2'1"
5'9"
12'6"
2'8"
13'10"
1'6"
1'6"
3'0"
3'0"
29'10"
9"
5'3"
19'0"
2'0"
9"
2'0"
3'
3"
9"
2'0"
SECTION A-A
61'0"
13'6"
34'0"
13'6"
6'1 1/2"
14'9"
19'3"
5'9"
14'9"
6'1 1/2"
2'1"
Sheet Piling
Construction Joint
Construction Joint
Anchor Bolts
3'7"
16'10"
3'7"
16'10"
61'0"
Center Line of Truss
Center Line between Trusses
Center Line of Truss
45'4"
2'1"
5'9"
Sheet Piling

FIG. 2.—DETAILS OF NORTH SHORE PIER

SUBSTRUCTURE

General Plan.—Exhaustive studies were made to determine whether to place the fixed shoes at one of the river piers, so that expansion would take place in both directions, or whether to place them at one end of the bridge.

The longitudinal force, resulting from traction, proved to be so great (2 200 000 lb) that when applied to either of the tall river piers it caused bending stresses that were too great to be resisted by a pier stem and footing of reasonable proportions and design. The point of application of such a force would have been 160 ft above the foundation bed of either river pier. It was decided, therefore, to fasten the bridge to the north shore pier and to provide roller shoes on the other three piers.

North Shore Pier.—The north shore pier is of unusual shape and large size, owing to the great longitudinal force it must withstand (see Fig. 2). It consists of a reinforced concrete footing, 18 ft thick, supporting reinforced concrete walls each 14 ft 9 in. thick at the base, and triangular in side elevation, constructed under each shoe. The longitudinal center line of each wall coincides with the center line of the truss it supports. These walls are connected and braced laterally by a reinforced concrete cross-wall, 13 ft 10 in. thick at the base and extending from the base to a plane flush with the tops of the side walls. The pier rests on 400 reinforced concrete piles, driven to refusal through the old fill into the underlying original ground. The term, "refusal," as used in this paper, denotes that the average set caused by each of the last five blows of the hammer should not exceed the amount determined by the following formula: $s = \frac{1.07\, wh}{P} - 0.1$, in which, s is the average final set, in inches, under each of the last five blows of the hammer; w, the weight of the falling parts of the hammer, in pounds; h, the drop of the hammer, in feet; and P, 60 000 lb for concrete piles as used. Reinforced concrete tongue-and-groove sheet-piling, 8 in. thick, was driven along the north and south sides of the foundation excavation, and the footing was cast between the walls thus formed. These walls were intended to assist in preventing the possibility of any horizontal movement of the pier.

The pier was designed as a cantilever beam that was considered to be fixed at the base. A force of 2 200 000 lb was applied 3 ft 4 in. above the top (the elevation of the keys in the shoes), acting in a line parallel to the center line of the bridge, in addition to the vertical reaction of the north end of the bridge. The resultant of these forces combined with the weight of the pier was kept well within the middle-third of the base to prevent subjecting any of the piles to uplift. Shears and moments were investigated at every critical section of each element of the pier. This pier contains 3 900 cu yd of concrete and 125 800 lb of reinforcing steel.

South Shore Pier.—The south shore pier is of usual design, consisting of a concrete footing with a reinforced concrete stem of rectangular cross-section. It rests on 325 reinforced concrete piles spaced on 3-ft centers in both directions. It contains 2 700 cu yd of concrete and 39 000 lb of reinforcement.

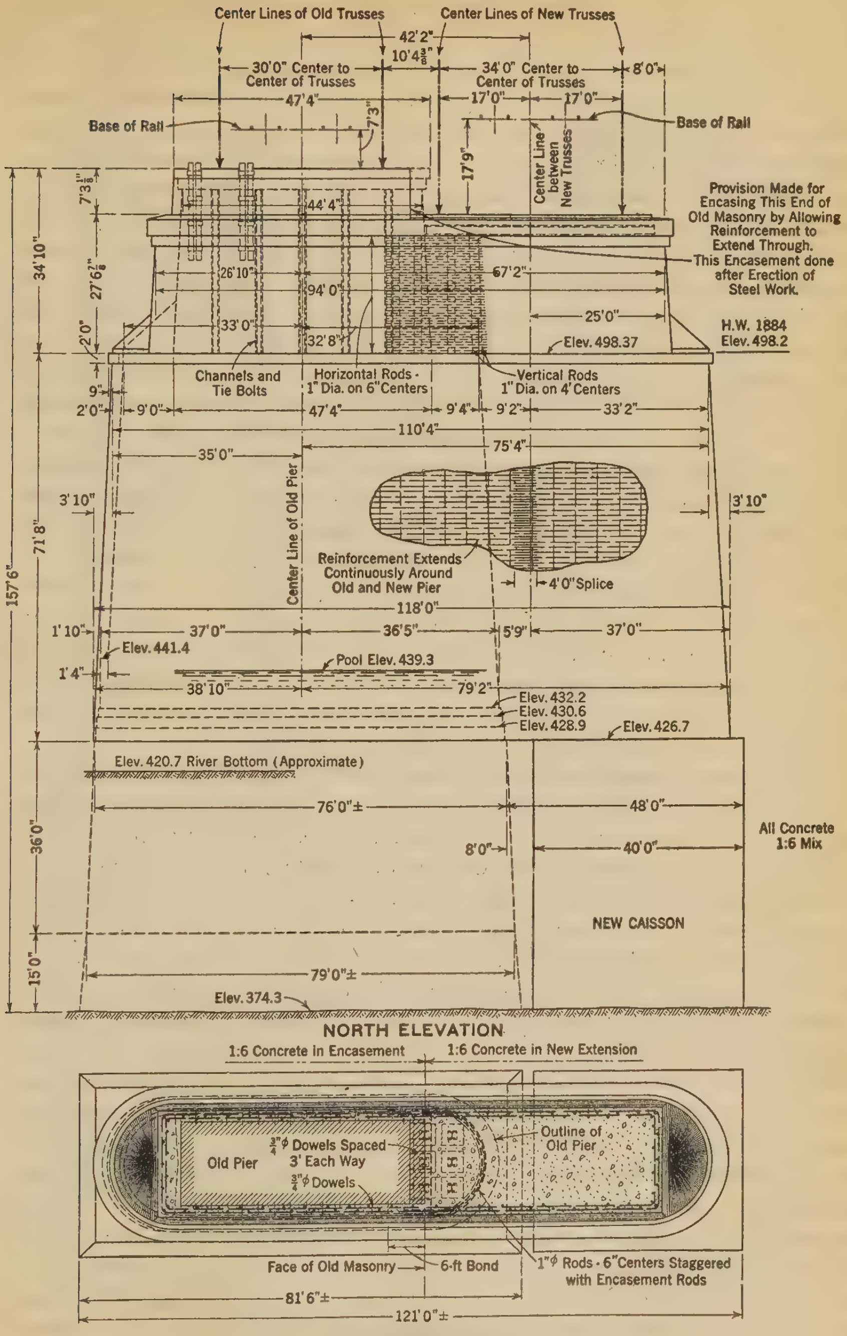
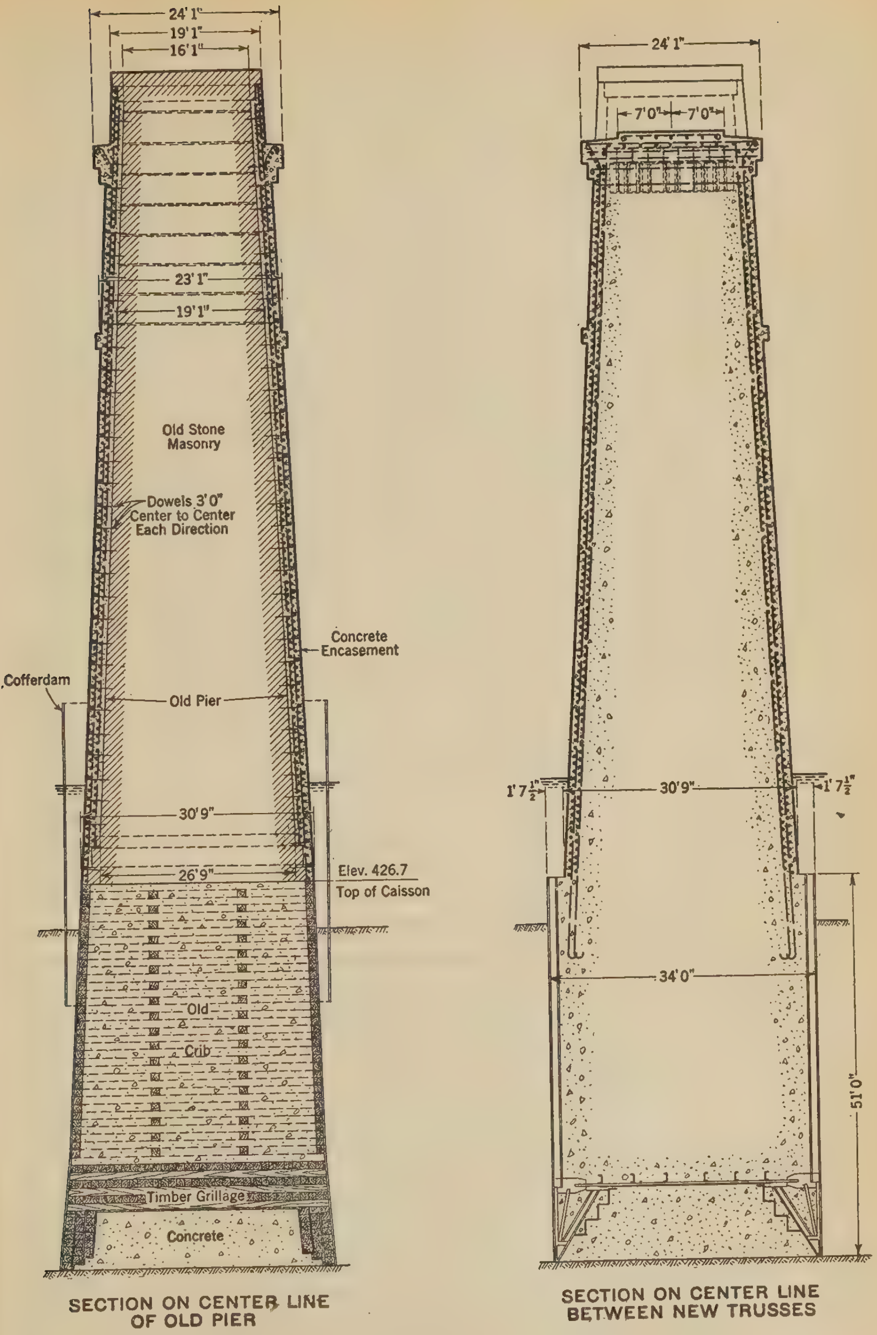

Center Lines of Old Trusses
Center Lines of New Trusses
42'2"
10'4$\frac{3}{8}$"
30'0" Center to Center of Trusses
34'0" Center to Center of Trusses
8'0"
47'4"
17'0"
17'0"
7'3"
Base of Rail
Base of Rail
17'9"
Center Line between New Trusses
44'4"
Provision Made for Encasing This End of Old Masonry by Allowing Reinforcement to Extend Through. This Encasement done after Erection of Steel Work.
26'10"
67'2"
94'0"
25'0"
33'0"
32'8"
H.W. 1884 Elev. 498.2
Elev. 498.37
7'3$\frac{1}{8}$"
27'6$\frac{7}{8}$"
2'0"
34'10"
Channels and Tie Bolts
Horizontal Rods - 1" Dia. on 6" Centers
Vertical Rods 1" Dia. on 4' Centers
9"
2'0"
9'0"
47'4"
9'4"
9'2"
33'2"
110'4"
75'4"
35'0"
3'10"
3'10"
Center Line of Old Pier
Reinforcement Extends Continuously Around Old and New Pier
4'0" Splice
71'8"
157'6"
118'0"
1'10"
37'0"
36'5"
5'9"
37'0"
Elev. 441.4
Pool Elev. 439.3
1'4"
38'10"
79'2"
Elev. 432.2
Elev. 430.6
Elev. 428.9
Elev. 426.7
Elev. 420.7 River Bottom (Approximate)
76'0"±
48'0"
All Concrete 1:6 Mix
8'0"
40'0"
36'0"
NEW CAISSON
15'0"
79'0"±
Elev. 374.3
NORTH ELEVATION
1:6 Concrete in Encasement
1:6 Concrete in New Extension
Old Pier
$\frac{3}{4}$"φ Dowels Spaced 3' Each Way
$\frac{3}{4}$"φ Dowels
Outline of Old Pier
Face of Old Masonry
6-ft Bond
1"φ Rods - 6" Centers Staggered with Encasement Rods
81'6"±
121'0"±
PLAN UNDER COPING

FIG. 3.—DETAILS OF SOUTH SHORE PIER

South River Pier.—The most interesting features of design and construction encountered in the substructure of the bridge were in connection with the south river pier. It was necessary to combine a new concrete extension on a concrete foundation with an old stone masonry pier resting on a timber foundation. The location of the steel superstructure is such that the down-stream shoe of the new bridge rests on the new extension, whereas the old pier must carry a large part of the reaction from the new up-stream shoe. This construction required the building of a 42-ft reinforced concrete extension, on a concrete caisson foundation, to the down-stream end of the old stone pier, which rests on a timber crib and timber caisson. The new extension was secured to the old pier as shown in Figs. 3 and 4. A concrete caisson was first sunk to rock foundation (Elevation 374.3). It was 34 ft wide and 40 ft long (parallel to the stream flow) and 25 ft high, and was constructed in the form of a steel shell of $\frac{1}{4}$-in. plate. It had a steel cutting-edge, a steel, plate-lined, working chamber, and two 7-ft dredging wells. The up-stream end of this caisson in its final position was approximately 8 ft from the down-stream end of the crib supporting the old pier. This steel shell, weighing 58 tons, was constructed on the north river bank, launched on October 6, 1927, and floated into position in its guide dock on October 17. Open dredging by a clam-shell bucket through the dredging wells was begun on November 1, 1927, and was continued intermittently (river stages permitting) until May 12, 1928, from which time sinking was continued by means of excavation in the working chamber, under compressed air. The caisson was landed on rock on May 21, 1928. After sufficient keys were cut in the rock foundation, it was sealed and the working chamber was filled with concrete.

A steel sheet-pile coffer-dam was then driven around both the old and the new foundations and unwatered to permit grouting and encasement of the old pier, and the construction of the concrete extension on the new caisson (see Fig. 4). The driving of this dam was seriously delayed by sunken logs and miscellaneous driftwood lodged against the northeastern (up-stream) corner of the old foundation. The caisson could not be unwatered until a 4-ft concrete seal had been poured, and this, with frequent high water, caused some further delay. By the time these difficulties were overcome, the steel superstructure had been erected to a point such that it semed doubtful whether the pier would be completed in time to receive the south side span which was being erected on falsework. The approach of the high-water season, with its attendant swift currents, and floating débris, made advisable the adoption of a plan to fabricate and erect a steel bent on the new foundation base of sufficient strength to support the south side span and to permit cantilever erection of the center span to its middle or closing point. It was proposed to leave this steel bent in place and incorporate it permanently in the body of the pier stem. The steelwork for this bent was fabricated, but was not needed because concerted effort on the part of the foundation contractor, as well as the fortunate occurrence of reasonably low water, permitted the pier to be completed in time to receive the side span.


24' 1"
19' 1"
16' 1"
23' 1"
19' 1"
Old Stone Masonry
Dowels 3' 0" Center to Center Each Direction
Concrete Encasement
Cofferdam
Old Pier
30' 9"
26' 9"
Elev. 426.7
Top of Caisson
Old
Crib
Timber Grillage
Concrete
24' 1"
7' 0"
7' 0"
1' 7$\frac{1}{2}$"
30' 9"
1' 7$\frac{1}{2}$"
34' 0"
51' 0"

SECTION ON CENTER LINE OF OLD PIER

SECTION ON CENTER LINE BETWEEN NEW TRUSSES

FIG. 4.—DETAILS OF SOUTH RIVER PIER

Prior to the encasement of the old pier stem its entire surface was thoroughly sand-blasted, holes were drilled through the outer shell into the masonry backing from each side, the joints of the masonry were pointed, and cement-sand grout was forced under pressure into the entire down-stream half of the stem in order to consolidate the masonry and fill all voids in the interior of the old pier. Only the down-stream half of the old pier was thus grouted because it was not considered that any of the load from the new bridge would be distributed to the up-stream half. The top stem, the up-stream end of which is split, was securely bound together by 15-in. channels placed vertically against the north and south faces and drawn tight by 2-in. rods placed in holes drilled through the pier stem. These channels were spaced 8 ft from center to center. Finally, the old clamps were removed.

When the grouting, cleaning, and pointing of the old pier, had been completed, a 2-ft reinforced concrete encasement was cast around it concurrently with the construction of the stem of the extension, and the old pier and the mass of the new extension were thoroughly bound together by horizontal steel reinforcement (see Figs. 3 and 4). All splices in the banding were made to develop the full value of the bars in tension, in order that the old and new work should be completely bound together by continuous steel reinforcement. The encasement of the old pier was secured to the stone masonry by expansion bolts set in the old stonework. These were of sufficient length to extend through the encasement to the plane of the outer row of horizontal reinforcing bars around which they were bent. These expansion bolts were spaced 3 ft, center to center, in both directions. It was anticipated that a vertical crack might develop in each face of the finished pier along the line where the encasement of the old pier joined the heavy mass concrete of the extension. Numerous plans for minimizing the effect and appearance of this anticipated crack were considered. It was finally decided to depend entirely on the horizontal reinforcement whereby the encasement and the stem of the extension are bound together. No such cracks or other evidences of the settlement have appeared to date (1933). The top stem of the old pier was not encased until after the new superstructure was finished because it was necessary to remove the down-stream end of the old top stem to make room for the up-stream shoe of the new bridge. The maximum working load on the foundation of the new extension is 13.4 tons per sq ft compared with 14.8 tons per sq ft on the foundation of the old pier.

North River Pier.—This pier consists of a reinforced concrete stem of the usual rectangular cross-section, with semi-cylindrical ends. It rests on a footing that was sunk in the form of a caisson. A steel shell, 84 ft long by 34 ft wide by 25 ft high, weighing 116 tons, was constructed on the north river bank. It was launched on September 24, 1927, and set in the guide dock on September 26. Sinking was begun on October 21, 1927, by open dredging through four tubes, 7 ft in diameter, located along its longitudinal center line. Between November 17, 1927, and January 10, 1928, little or no progress was made because of high-water stages. From about the middle of January, 1928, the work progressed almost continuously until May 28, 1928, at which time locks were erected on the tubes and sinking was continued by the pneumatic method. At the time air was applied the cutting-edge was approximately 7 ft above the final rock foundation, upon which the caisson was landed on June 9, 1928. Due to high-river stages (several of which necessitated temporary suspension of work in the caisson), the caisson was not sealed nor the working chamber filled until July, 1928.

Construction Plant and Methods.—The contractor's plant for the substructure consisted essentially of a tower on the north bank of the river, a mixing plant at the base of the tower, and a narrow-gauge track for a gasoline-propelled train of concrete buggies on the vehicular roadway along the down-stream side of the old bridge. The mixing plant consisted of the usual sand and gravel bins, water tank, and a 1-yd mixer. Concrete was made and distributed to both shore piers and the north river pier by means of this plant. A floating plant was used for making and placing concrete in the south river pier, because when this pier was being poured the down-stream roadway was being removed from the old bridge to permit steel erection. In order to facilitate concreting the north river pier, the concrete tower was moved from the north bank to a location in the river near that pier. It was supported on piles driven in the river and was securely fastened to the old bridge.

All concrete was made in the proportions of 1 part cement to 6 parts aggregate, with the sand and gravel measured separately. A combination of 2.3 parts of sand with 3.7 parts of gravel was found to produce concrete of maximum density which, when mixed for not less than 1 min in the mixer with from $5\frac{3}{4}$ to 6 gal of water per sack of cement, attained an average strength at 28 days of about 2 500 lb per sq in., as indicated by cylinder tests. Three pounds of an admixture of diatomaceous silica per bag of cement was used, and this assisted materially in producing concrete of satisfactory workability.

SUPERSTRUCTURE

General Description.—Numerous studies were made to determine the proper character and lines for the 1 575-ft, continuous truss. Consideration of appearance, as well as economy, resulted in the selection of a triangular, or Warren, type with sloping top chords and subdivided panels. The trusses are 34 ft from center to center, with top-chord cover-plates 44 in. wide, thus providing adequate railroad clearance and making the distance from center to center of trusses slightly more than one-twentieth of the length of the center span. Other dimensions and the grades of roadway, are shown in Fig. 1.

Silicon alloy steel was used throughout at a basic working stress of 24 000 lb per sq in. for all steelwork, except rivets. The resultant savings in steel weight was approximately 19% of the weight of a similar structure built of structural grade carbon steel at 18 000 lb per sq in. Movable shoes on steel rollers, 24 in. in diameter, enclosed in oil boxes, were provided on the south shore pier and on both river piers.

The silicon steel was manufactured by the open-hearth process according to specifications which required the following chemical properties:

	Ladle analysis	Check analysis
Carbon, maximum percentage............	0.40	0.44
Phosphorous, maximum percentage:		
Acid	0.06	0.075
Basic	0.04	0.05
Sulfur, maximum percentage............	0.05	0.063
Silicon, minimum percentage............	0.20	0.18

The specified ultimate strength was 80 000 to 95 000 lb per sq in., with a minimum yield point of 45 000 lb per sq in. A review of the tests shows that the elastic limit of the material obtained was consistently from 2 000 to 4 000 lb per sq in. in excess of the minimum requirement, with silicon content ranging between 0.2 and 0.3 per cent.

Design Loads and Stresses.—Cooper's E-70 engine loading was used in computing the live load stresses in the floor system, hangers, and subdiagonals. The impact increments added to stresses thus determined were based on the formula:

$$I = S\ \frac{2\,000 - L}{1\,600 + 10\,L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, I = impact increment; S = computed maximum live load stress, in pounds per square inch; and L = loaded length of track, in feet, producing the maximum live load stress in the member. This includes the length of both tracks, loaded, for members receiving a maximum live load stress for this condition.

The trusses were designed for 90% of Class E-70 engine loading, which for the purpose of computing stresses was represented by a load of 6 300 lb per ft of truss, with a load of 95 000 lb concentrated at each of two panel points in each truss, separated by two panel lengths. In determining maximum stresses, full consideration was given to the effect of split loads using the aforementioned concentrations.

Impact increments of the live load stresses in main truss members were found by using the following formulas:

For stresses of single character:

$$I = \frac{300}{300 + L} \times \frac{\text{Live Load}}{\text{Live Load} + \text{Dead Load}} \quad \ldots\ldots\ldots\ldots (2)$$

For reversal of stresses:

$$I = \frac{300}{300 + L} \times \left(1 + \frac{m}{2\,M}\right) \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, M = the maximum algebraic sum of dead and live load stresses of like character; and m = the maximum algebraic sum of dead and live load stresses of opposite character. Equation (3) was not used when the static live load was less than the dead load stress of opposite character.

Lateral forces from trains and wind of 1 100 lb per lin ft on the loaded chord, plus 10% of the specified train load on one track, and 400 lb per lin ft on the unloaded chord, all considered as moving loads, were used.

Longitudinal forces were determined by providing for the effect of starting and stopping trains with a coefficient of friction of 20% on engine drivers and of 10% on the remainder of the train. Members subject to stress reversal were proportioned to resist the maximum stress of either character. The permissible maximum unit stresses used were as follows, in pounds per square inch:

Axial tension on net section (silicon steel)	24 000
Axial compression on gross section of columns (silicon steel)	$\dfrac{24\,000}{1+\dfrac{l^2}{9\,000r^2}}$
with maximum of in which, l is the length of the member, in inches, and r is the least radius of gyration.	20 400
Direct compression on steel castings	18 000
Bending:	
In extreme fibers of rolled shapes, built sections, and girders, net section (silicon steel)	24 000
In extreme fibers of pins (silicon steel)	36 000
Shearing:	
Webs, gross section (silicon steel)	15 000
Shop-driven rivets (rivet steel)	12 000
Field-driven rivets, fitted bolts, and pins (rivet steel)	10 000
Bearing:	
Shop-driven rivets, projected diameter (rivet steel)	24 000
Field-driven rivets, fitted bolts, and pins, projected diameter (rivet steel)	20 000
On masonry or concrete	600
Expansion rollers per linear inch (silicon steel). in which, d is the diameter of the roller, in inches.	$600d$

Proportioning of Parts.—The proportioning of the various parts and all details were made generally consistent with the Specifications for Design and Construction of Steel Railway Bridge Superstructures.[4]

Method of Design.—The theory of elastic deformation was used in determining all stresses. For convenience in determining conditions of loading required to produce maximum stresses and the stresses themselves, influence lines (Fig. 5) were used. The first set of influence lines prepared for the end and intermediate reactions were based upon the assumption that the moment of inertia of the truss throughout its length would be constant.

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[4] Final Report of the Special Committee on Specifications for Bridge Design and Construction, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 471 *et seq.*

Members of the trusses were proportioned for the stresses resulting from these reactions. By use of the theory of elastic weights, adjustments were then made in the influence lines to give effect to the variation of the moment of inertia which actually prevails, and the adjusted influence lines as shown on Fig. 5 were used in the final design. Table 1 shows a comparison of the ordinates of the influence lines based on constant moment of inertia with corresponding ordinates of the adjusted curves.

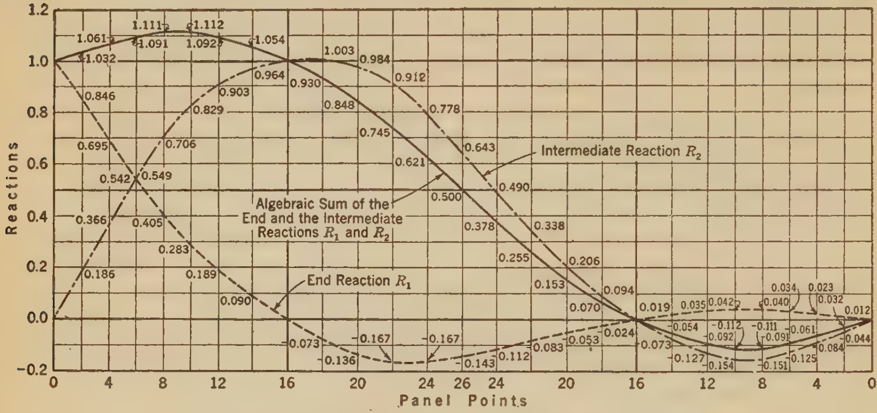


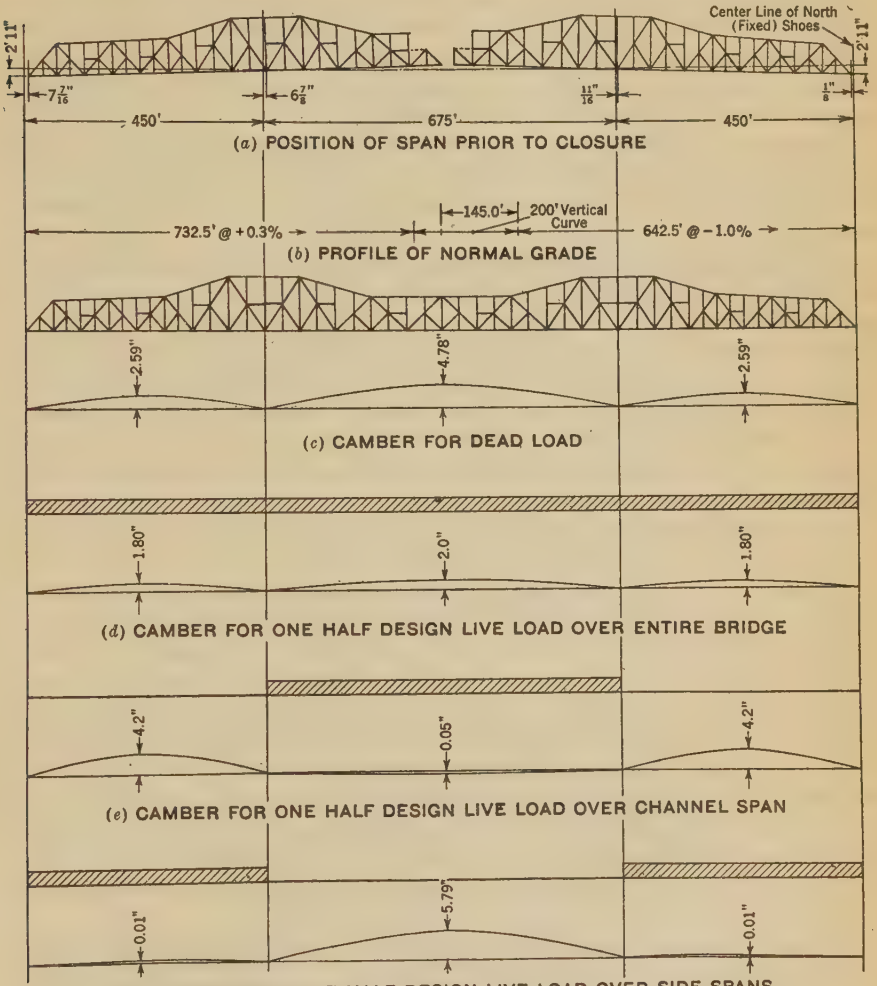
FIG. 5.—INFLUENCE LINES (FOR PANEL POINTS SEE FIG. 1)

Erection stresses in no case governed the design of any member although they were based on cantilever erection of the middle span with no falsework between the channel piers. (Stresses, caused by erection plant, 25% in excess of dead plus live load stresses were permissible.) Therefore, this method of erection did not necessitate the addition of any steel to that required in the finished structure for resisting dead, live, impact, and wind loads.

TABLE 1.—COMPARISONS OF INFLUENCE LINES

Panel Point No. (See Fig. 5)	END REACTION, R_1		INTERMEDIATE REACTION, R_2		Panel Point No. (See Fig. 5)	END REACTION, R_1		INTERMEDIATE REACTION, R_2	
	Constant moment of inertia	Variable moment of inertia	Constant moment of inertia	Variable moment of inertia		Constant moment of inertia	Variable moment of inertia	Constant moment of inertia	Variable moment of inertia
0......	1.000	1.000	0.00	0.00	24.....	—0.109	—0.112	0.488	0.490
2......	0.853	0.846	0.164	0.186	22.....	—0.082	—0.083	0.346	0.338
4......	0.714	0.695	0.324	0.366	20.....	—0.051	—0.053	0.211	0.206
6......	0.583	0.542	0.476	0.549	18.....	—0.024	—0.024	0.091	0.094
8......	0.452	0.405	0.616	0.706	16.....	0.0	0.0	0.000	0
10......	0.332	0.283	0.741	0.829	14.....	0.016	0.019	—0.060	—0.073
12......	0.223	0.189	0.846	0.903	12.....	0.024	0.035	—0.091	—0.127
14......	0.096	0.090	0.947	0.964	10.....	0.025	0.042	—0.098	—0.154
16......	0.0	0.0	1.000	1.000	8.....	0.024	0.040	—0.094	—0.151
18......	—0.070	—0.073	1.006	1.003	6.....	0.021	0.034	—0.079	—0.125
20......	—0.114	—0.136	0.955	0.984	4.....	0.015	0.023	—0.056	—0.084
22......	—0.136	—0.167	0.872	0.912	2.....	0.008	0.012	—0.029	—0.044
24......	—0.138	—0.167	0.760	0.788	0.....	0	0	—0.000	0
26......	—0.130	—0.143	0.629	0.643					

The spans of the bridge were cambered so that the center line of the bottom chord would not sag below a line parallel to the normal grade line, as shown on Fig. 6, when the entire bridge or each span thereof is subjected to one-half the specified live load. This camber was obtained by correcting the lengths of the top and bottom chord members in accordance with the deformations produced in them and fabricating the web members to their geometrical lengths in the resulting truss figure. The trusses could thus be


2'11"
Center Line of North (Fixed) Shoes
2'11"
7 7/16"
6 7/8"
11/16"
1/8"
450'
675'
450'
145.0'
200' Vertical Curve
732.5' @ +0.3%
642.5' @ −1.0%
2.59"
4.78"
2.59"
1.80"
2.0"
1.80"
4.2"
0.05"
4.2"
0.01"
5.79"
0.01"

(*a*) POSITION OF SPAN PRIOR TO CLOSURE

(*b*) PROFILE OF NORMAL GRADE

(*c*) CAMBER FOR DEAD LOAD

(*d*) CAMBER FOR ONE HALF DESIGN LIVE LOAD OVER ENTIRE BRIDGE

(*e*) CAMBER FOR ONE HALF DESIGN LIVE LOAD OVER CHANNEL SPAN

(*f*) CAMBER FOR ONE HALF DESIGN LIVE LOAD OVER SIDE SPANS

FIG. 6.—ERECTION AND CAMBER DIAGRAMS

erected on falsework with blocking set to conform to their shapes under no load, without forcing the connections except as hereinafter noted. In order to reduce the dips in the cambered grade over the channel piers (see Fig. 6), the floor-beams at these two points were placed ½ in. higher above the center lines of the bottom chords than all other floor-beams.

Secondary stresses were avoided wherever possible, but were recognized and provided for in the design where their complete elimination was not feasible. It is evident that without special provision the heaviest secondary stresses would occur in the bottom chords and inclined posts framing into the connections at the panel points over the channel piers. Secondary stresses in the bottom chords at these points were reduced materially by making the sub-verticals on each side of these panel points $\frac{1}{2}$ in. shorter than the detailed length required for camber of the span, and forcing the chords to connections therewith, thereby inducing initial stresses in the chords and sub-verticals that were relieved when the bridge was swung and under full dead load. The inclined posts were connected to the gusset-plates by pins, thereby eliminating secondary stresses entirely from these members.

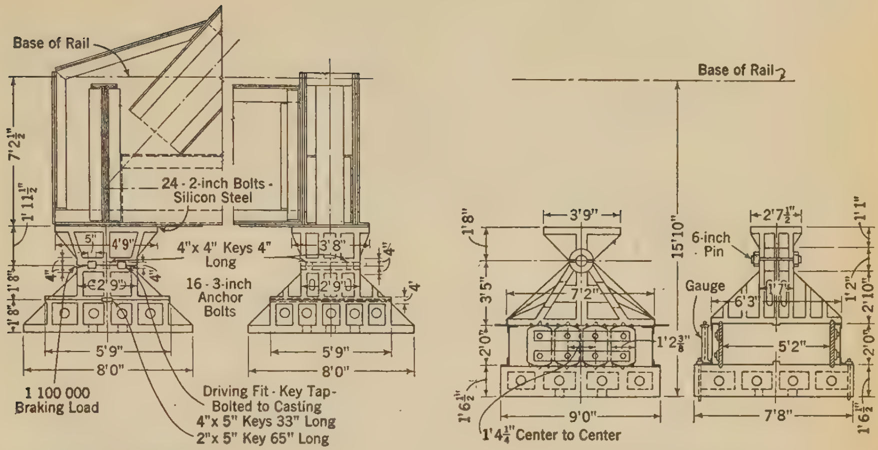
The bottom lateral system was designed as a continuous truss of three spans. The top lateral system was designed as simple spans between the end hip connections and the upper ends of the inclined posts at the two channel piers.

Bearings.—Fig. 7 illustrates the details of the bearing shoes, which are of cast steel. The rollers are of rolled steel billets and the anchor-bolts are structural carbon steel. Each of the fixed shoes on the north shore pier consists of a single top casting fastened to the bottom chord of the truss, and a bottom part consisting of two castings. The upper castings attached to the trusses are locked to the base part by the rectangular keys (4 by 5 in. in cross-section, and 33 in. long) shown in Fig. 7(*a*), and these keys withstand the longitudinal force caused by starting and stopping trains on the bridge. The lower surface of each top casting is finished to a perfect plane, while the top surface of the middle casting on which the top casting rests, was finished to a radius of 1 200 in. between keys and 600 in. outside of keys, thus eliminating the possibility of serious edge pressures in the castings due to live load deflection of the side span. Each of the two fixed shoes weighs 18.8 tons.

Each of the shoes on the two river piers and on the south shore pier (Fig. 7(*b*) and Fig. 7(*c*)) consists of a base casting supporting steel rollers (24 in. in diameter) on which the upper part of the shoe rests. In the case of the shoes on the river piers (Fig. 7(*b*)), this upper part consists of two castings, one upon the other, while in the case of the shoe on the south shore pier (Fig. 7(*c*)), it consists of a single casting. The castings attached to the bottom chords are supported on pins, $11\frac{1}{2}$ in. in diameter, which are carried by the upper bases of the shoes. The total weight of one shoe on each river pier is 96.5 tons, and the total weight of each shoe on the south shore pier is 38.5 tons.

To prevent corrosion of the rollers and bearing surfaces, and accumulation of débris, steel boxes were constructed around the rollers, and on the completion of the bridge were filled with an unadulterated, petroleum-base cylinder oil. Approximately, 1 600 gal of oil were required to fill the six bearings.

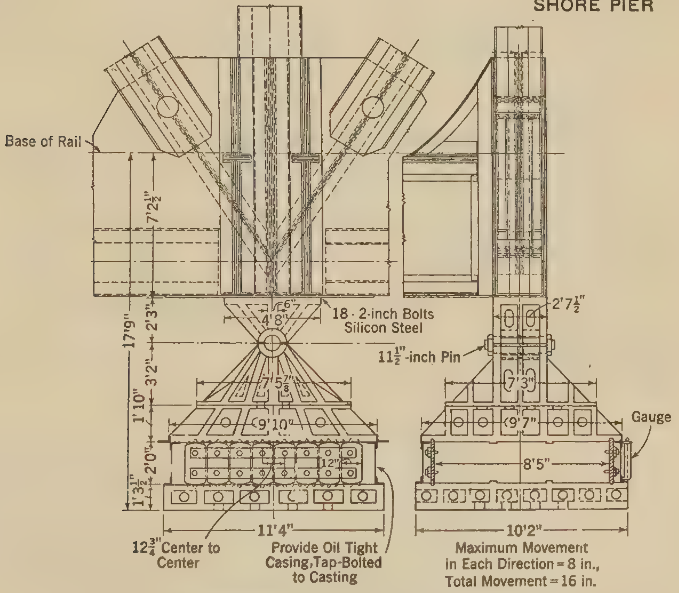
Floor System and Track Deck.—The floor system consists of four lines of plate girder stringers framing into the floor-beams. At the panel points over the channel piers, double floor-beams were used to serve as jacking beams for the adjustment of the dead load reactions prior to putting the bridge in service and for any subsequent adjustment that might be required. (See Fig. 8.) The bottom flanges of each of these floor-beams and those at the


Base of Rail
7'2½"
1'11½"
24 - 2-inch Bolts - Silicon Steel
5"
4'9"
4"x 4" Keys 4" Long
3'8"
4"
4"
4"
16 - 3-inch Anchor Bolts
2'9"
2'9"
4"
1'8"
1'8"
5'9"
8'0"
5'9"
8'0"
1 100 000 Braking Load
Driving Fit - Key Tap-Bolted to Casting
4"x 5" Keys 33" Long
2"x 5" Key 65" Long
Base of Rail
3'9"
2'7½"
1'1"
1'8"
15'10"
6-inch Pin
3'5"
7'2"
Gauge
1'7"
6'3"
1'2"
2'10"
2'0"
1'2⅜"
5'2"
2'0"
1'6½"
9'0"
7'8"
1'6½"
1'4¼" Center to Center

(*a*) FIXED SHOE, OHIO SHORE PIER

Maximum Movement in Each Direction = 11 in., Total Movement = 22 in.

(*c*) EXPANSION SHOE, KENTUCKY SHORE PIER


Base of Rail
7'2½"
6"
4'8"
18 - 2-inch Bolts Silicon Steel
17'9"
2'3"
2'7½"
11½"-inch Pin
3'2"
7'5⅞"
7'3"
1'10"
9'10"
9'7"
Gauge
2'0"
12"
8'5"
1'3½"
11'4"
10'2"
12¾" Center to Center
Provide Oil Tight Casing, Tap-Bolted to Casting
Maximum Movement in Each Direction = 8 in., Total Movement = 16 in.

(*b*) EXPANSION SHOE, INTERMEDIATE PIER

FIG. 7.—DETAILS OF SHOES

ends of the bridge were reinforced to provide seats for the application of the jacking load under the center lines of the exterior stringers. These double floor-beams, together with sliding bearings for stringers at the sixth panel point from the channel pier in each end of the middle span, serve to break the deck into independent sections and thereby to prevent induced stresses in the stringers as a result of chord elongation caused by deflection of the trusses under live load. The stringers are braced by the usual Warren type bracing in the plane of their top flanges. The members of the bottom lateral bracing of the bridge are in the plane of the bottom flanges of the stringers and are connected to them. The floor-beams are stiffened by braking trusses which lie in the plane of the bottom flanges of the stringers to which they are connected.

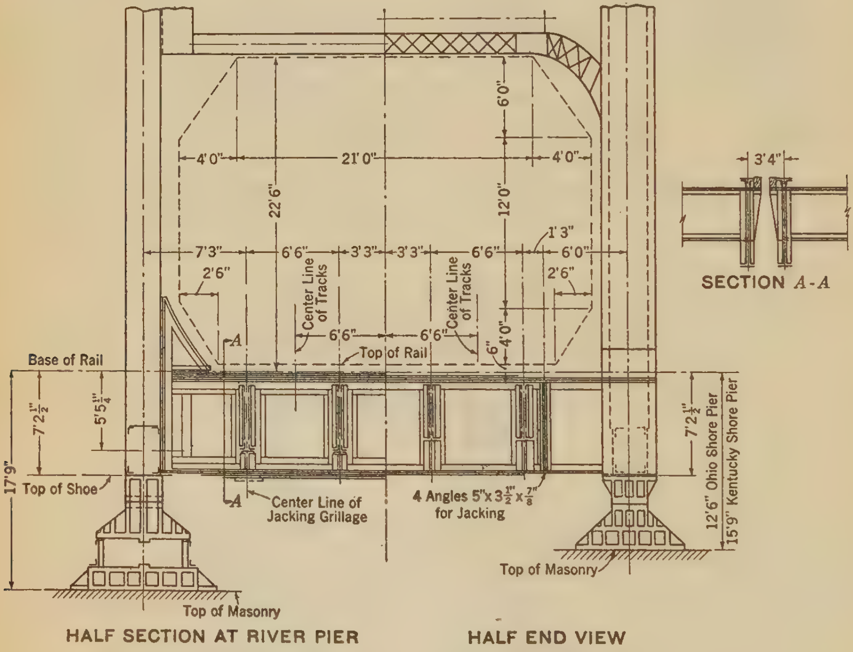

4'0"
21'0"
4'0"
6'0"
22'6"
12'0"
1'3"
7'3"
6'6"
3'3"
3'3"
6'6"
6'0"
2'6"
2'6"
Center Line of Tracks
Center Line of Tracks
6'6"
6'6"
A
Top of Rail
6"
4'0"
Base of Rail
7'2½"
5'5¼"
17'9"
Top of Shoe
A
Center Line of Jacking Grillage
4 Angles 5"x3½"x⅞" for Jacking
Top of Masonry
Top of Masonry
7'2½"
12'6" Ohio Shore Pier
15'9" Kentucky Shore Pier
3'4"
SECTION A-A
HALF SECTION AT RIVER PIER
HALF END VIEW

FIG. 8.—SECTION VIEWS OF ROADWAY

At the south end of the bridge, provision was made for maximum movement, due to thermal expansion and contraction, of 22 in. In order to provide support for the track deck at this point a short floating beam was placed between the end floor-beam of the bridge and the end floor-beam of the plate girder approach. It was supported on sliding bearings attached to each of these floor-beams, as shown in Fig. 9. These bearings are fitted with stop angles so as to permit 12 in. of movement of the floating beam on its seat at each end, thus providing for a total movement of 24 in. between the end of the main bridge and the plate girder approach. The floating beam carries four 8 by 12-in. cross-ties, between which spacer blocks were placed, to prevent them from bunching as a result of the movement of the end of the bridge. The track deck consists of 8 by 10-in. creosoted cross-ties, 130-lb **T**-rail, guard-rails, and a plank walkway between the tracks. The rails were laid continuous over the bridge and approaches, with no special frog or other device at the south end of the bridge to provide for expansion movement. The superstructure contains 16 275 000 lb of silicon steel, exclusive of cast-steel shoes and rollers, the aggregate weight of which is 1 000 000 lb.

Erection of the Superstructure.—The side spans were erected on falsework constructed on piles driven in the river. The middle span was erected by cantilevering over both channel piers to closure at its center.

All the floor-system material for the parts of the two side spans over the north and south river banks was shipped to the site by rail and erected on the falsework by means of locomotive cranes operating on the bridge deck. Arrangements had been made for the Railroad Company to complete the viaduct approaches to the ends of the bridge before the erection contractor began on the floor system, and this greatly facilitated the work.

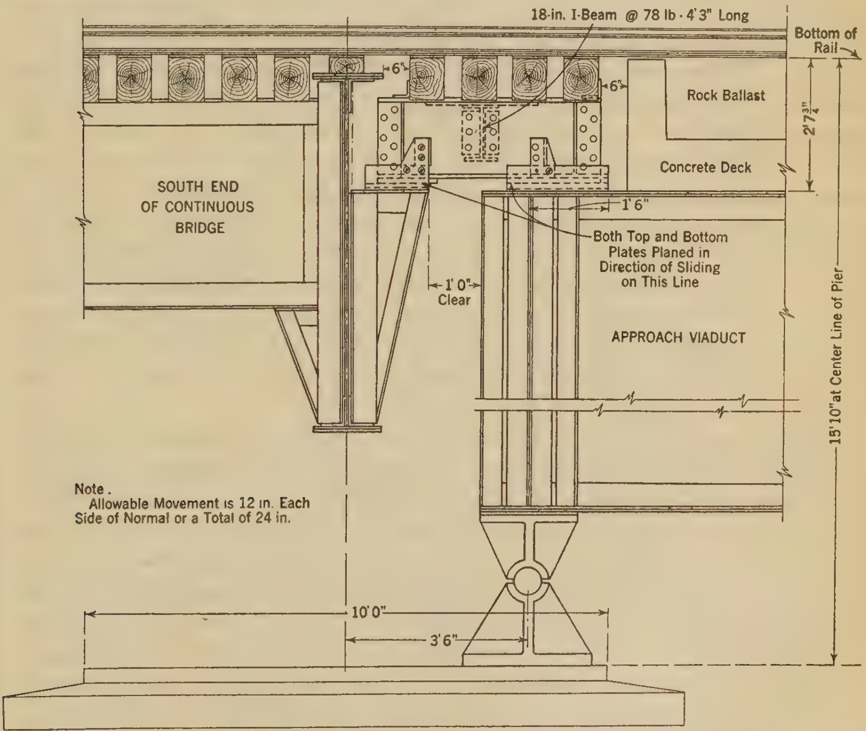

18-in. I-Beam @ 78 lb · 4'3" Long
Bottom of Rail
6"
6"
Rock Ballast
2'7¾"
Concrete Deck
SOUTH END OF CONTINUOUS BRIDGE
1'6"
Both Top and Bottom Plates Planed in Direction of Sliding on This Line
1'0" Clear
APPROACH VIADUCT
15'10" at Center Line of Pier
Note. Allowable Movement is 12 in. Each Side of Normal or a Total of 24 in.
10'0"
3'6"

FIG. 9.—SECTION OF EXPANSION JOINT AT SOUTH END OF BRIDGE

All materials for trusses, bracing, center-span floor system, and floor system for the parts of the side spans situated over the water, were shipped to the site by water and erected from barges with the help of locomotive cranes running on the bridge deck. The heaviest single piece handled weighed 34 tons and consisted of a floor-beam with four stringers attached. Both side spans were erected simultaneously. There was a time, in the early fall of 1928, when serious anxiety was felt for the safety of the south side span, due to delay in the completion of the south channel pier. It appeared that in order to prevent subjecting the span on falsework to the hazards of the high-water months, including swift flood currents, and floating ice and débris, that either the span must be dismantled and stored through the winter or the steel bent or tower, previously described under "South River Pier," on which to land the span, must be erected. The latter plan was adopted, and the tower was fabricated but, as previously stated, its erection was not required. As a precaution, heavy timber piles were driven up stream of each bent of the falsework, and the falsework was lashed to them by steel wire cables in such a manner as to moor it against the force of a flood and also to protect it to some extent from floating débris. The span was completed and swung without mishap.

An interesting detail in connection with the erection of this side span was that caused by the necessity of removing a part of the top stem of the old south river pier enough to clear the up-stream shoe of the new bridge (see Fig. 10). The importance and volume of railroad traffic carried on the old bridge during the erection of the new bridge made it impossible to abandon the use of either track even for a short time. As the part to be cut from the down-stream end of the top stem of the old pier extended to a vertical plane tangent to the down-stream end of the shoe of the old bridge, and as the structural value of the masonry of the old pier was more or less unknown (even though grout had been pumped into it), some anxiety was felt for the safety of the southbound track. The operation was accomplished safely by careful work in removing only as much of the old pier stem as was necessary to clear the new shoe. The masonry of the inshore and river faces of the old stem was left in place and served as buttresses against the shearing of the top edge of the old pier.

While the bridge was being erected the fixed shoes were omitted, and jacks were placed on the north shore pier under the ends of the trusses. This end of the bridge was shifted $\frac{1}{8}$ in. south and held 2 ft 11 in. below its final position. The centers of the shoes on the north channel pier were shifted $\frac{11}{16}$ in., and the centers of the shoes on the south channel pier were shifted $6\frac{7}{8}$ in., north of the centers of bearings on their respective piers, and the rollers of the shoes on both piers were locked. The south end of the bridge was set so that the end panel point was $7\frac{7}{16}$ in. north of the center of bearing on the pier; the shoe above the rockers was omitted; and temporary blocking was placed between the chord and the rollers which were locked so as to maintain the chord 2 ft 11 in. below its final position. At closure, the bottom chord connection was made, after which the north end of the bridge was jacked up until the top chord closed. The position of the bridge prior to closure is shown on Fig. 6. The shoes at both ends of the bridge were erected and the lock-bars were removed from the rollers of the movable shoes, after the bridge had closed.

The final step in erection consisted of "weighing" the bridge by jacking up the bearings on the two shore piers until the predetermined dead load reaction (of 617 200 lb) on each pier was attained, as shown by pressure

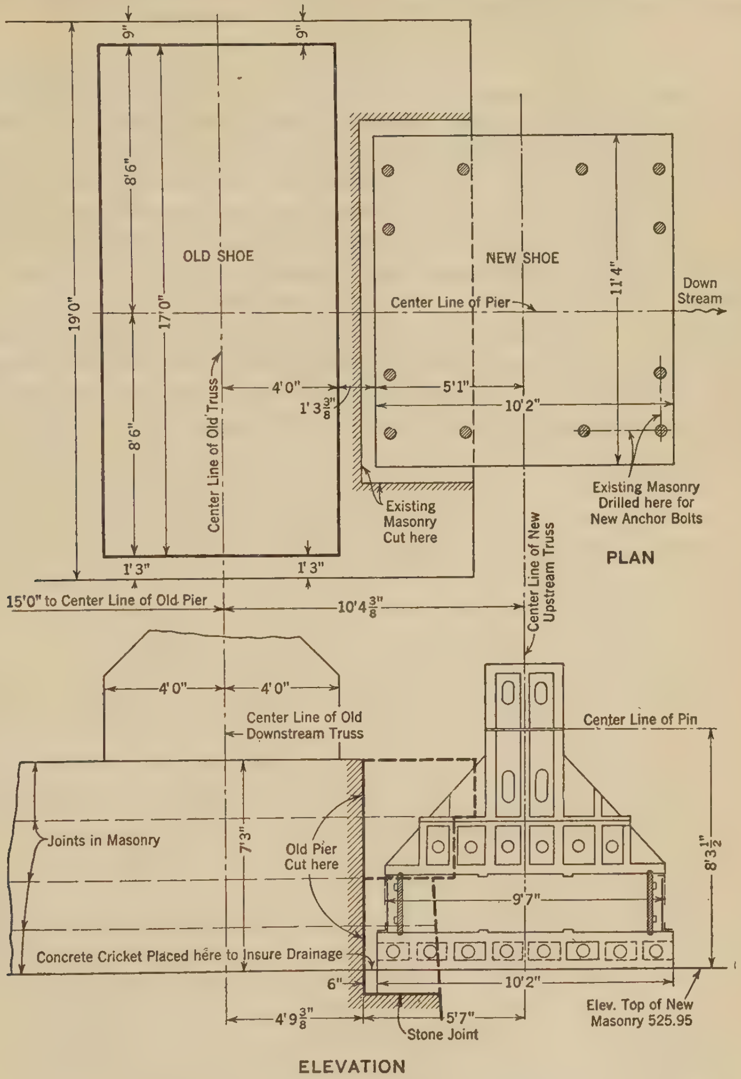

9"
9"
19'0"
17'0"
8'6"
8'6"
OLD SHOE
NEW SHOE
Center Line of Old Truss
4'0"
1'3$\frac{3}{8}$"
5'1"
10'2"
11'4"
Center Line of Pier
Down Stream
Existing Masonry Cut here
Existing Masonry Drilled here for New Anchor Bolts
PLAN
1'3"
1'3"
15'0" to Center Line of Old Pier
10'4$\frac{3}{8}$"
Center Line of New Upstream Truss
4'0"
4'0"
Center Line of Old Downstream Truss
Center Line of Pin
Joints in Masonry
7'3"
Old Pier Cut here
9'7"
8'3$\frac{1}{2}$"
Concrete Cricket Placed here to Insure Drainage
6"
10'2"
4'9$\frac{3}{8}$"
5'7"
Stone Joint
Elev. Top of New Masonry 525.95
ELEVATION

FIG. 10.—DETAILS OF CUT MADE IN OLD MASONRY TOP STEM OF SOUTH RIVER PIER TO PROVIDE ROOM FOR NEW SHOE

gauges on the jacks. Where necessary, shims were placed between the underside of the bottom chords and the shoe castings in order to maintain the bridge in the form producing the aforementioned dead load reaction. The position of the movable shoes was checked and found to be correct, and the bridge was ready for service.

CONCLUSION

The continuous bridge is a type that should receive more consideration than has been accorded it in America. It is particularly well suited to railroad service because of its great rigidity.

ACKNOWLEDGMENTS

The United Gas Improvement Contracting Company, of Philadelphia, Pa., built the piers, and the American Bridge Company furnished and erected the steel superstructure. The bridge was designed by The J. E. Greiner Company, which served in a consulting capacity during the construction, and was built under the supervision of C. W. Johns, M. Am. Soc. C. E., and Crosby Miller, Assoc. M. Am. Soc. C. E., Chief Engineer and Bridge Engineer, respectively, of the Chesapeake and Ohio Railway Company, with Mr. G. G. Lancaster, of the Railway Company, as Resident Engineer.

Construction of the substructure was begun in July, 1927, and completed in October, 1928. Work was often interrupted by abnormally frequent and prolonged periods of high water during the spring and summer of 1928. Steel erection was begun in June, 1928, and the bridge was put in service in March, 1929.

DISCUSSION

J. E. WILLOUGHBY,[5] M. AM. SOC. C. E. (by letter).—The plans for, and construction of, the south river pier of the bridge described by Mr. Ballard are of much interest to those who are responsible for the reconstruction of existing bridges. Definite economies result from the preservation of existing masonry in bridge reconstruction projects, and the author has described a successful engineering solution of a specific problem for a great bridge. It is to be regretted that the requirements of the War Department prevented similar extension construction for the north river pier, with its resulting economy to the users of transport. Now that the author has proved the case with regard to the substructure of a great bridge, engineers may expect the adoption of his methods for future similar projects, thus furthering the economy of bridge reconstruction. It is a prevailing error of those in charge of reconstruction to cast away much of the existing works instead of incorporating them into the rebuilt structure. For railway bridges, the elaborate accounting system prescribed by public authority adds to the waste.

For many years, the writer has endeavored to preserve the existing masonry in the bridge reconstruction projects of railway bridges. While those projects have not been of magnitude much money has been saved to the owners of the bridges. A late project was the strengthening of the cast-iron cylinders that formed a pier for a railway bridge over the Cape Fear River, near Wilmington, N. C. The pier was built in 1868–69, and consisted of two cast-iron cylinders, each 96 in. in diameter, founded on marl 50 ft below mean low tide. The pier supported one end of a through truss fixed span, 216 ft in length, and served as a rest pier for a rolling-lift draw span, 118 ft in length. The steps to strengthen the pier and to give it added bearing area on the marl were, to:

(1) Clear, by dredging, the accumulation of riprap, logs, silt, and sand from around both cylinders, the sunken logs being removed with locomotive crane;

(2) Construct about the two cylinders a shell of $\frac{3}{8}$-in. steel plate, assembled in the field, section by section, above mean tide, and then lowered;

(3) Pump and jet from the interior of the shell, and around the outside, so as to free the shell for further lowering, until the bottom of the river, 43 ft below mean tide, was reached by the lower edge of the shell;

(4) Remove, by pumping, all loose sand and débris from the marl in the interior of the shell, when the lower edge of the shell has reached that depth;

(5) Place a seal of concrete through the tremie, and add concrete above the seal sufficient to prevent displacement and distortion of the shell by the bouyancy and pressure of the water outside; and

(6) Unwater the shell and complete the concrete encasement of the two cylinders.

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[5] Chf. Engr., A. C. L. R. R., Wilmington, N. C.

WILSON T. BALLARD,[6] M. AM. SOC. C. E. (by letter).—The writer appreciates the discussion offered by Mr. Willoughby. It adds to the value of the original paper. He agrees with Mr. Willoughby that wherever possible the original investment in existing structures should be conserved to the fullest extent when reconstruction of such structures is undertaken.

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[6] Vice-Pres., The J. E. Greiner Co., Baltimore, Md.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1877

ACTUAL DEFLECTIONS AND TEMPERATURES IN A TRIAL-LOAD ARCH DAM

By A. T. Larned[1] and W. S. Merrill[2], Members, Am. Soc. C. E.

With Discussion by Messrs. George Jacob Davis, Jr., D. C. Henny, B. E. Torpen, Lars R. Jorgensen, H. B. Muckleston, Ivan E. Houk, A. V. Karpov and R. L. Templin, and A. T. Larned and W. S. Merrill.

Synopsis

This paper contains a description of the layout of the dam of the Ariel Hydro-Electric Development, and the construction methods used in building it; but the following four points are emphasized principally:

1.—The design of the arch dam, and the effect of the cement content and the manner and speed of construction on its thickness and the resulting stresses;

2.—The proportioning of the concrete to secure good strength with a minimum of setting-heat generation;

3.—The amount of heat generated in the concrete mass during setting and how it was dissipated with time; and,

4.—The radial and tangential deflections of the arch.

It is shown that the amount of heat generated in a concrete mass can be calculated closely so that the construction methods and schedule can be arranged accordingly. If necessary the cooling can be accelerated without harm to the concrete. It is also shown that the concrete in the interior of a mass, where the setting heat is greatest, is as strong as, or stronger than, that nearer the surface. There is a satisfactory agreement between the actual and the calculated radial and tangential deflections.

Introduction

The Ariel Hydro-Electric Development is on the north fork of the Lewis River, at Ariel, Wash., about 25 miles north of Portland, Ore. The location is clearly indicated in Fig. 1. The initial installation consists of one

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Note.—Published in May, 1933, *Proceedings*.

[1] Engr., Elec. Bond and Share Co., New York, N. Y.

[2] Hydr. Engr., Elec. Bond and Share Co., New York, N. Y.

45 000-kw unit under a static head, with reservoir full, of 185 ft. The power house is situated in and over the old stream bed immediately below the arch portion of the dam. The arch dam has a maximum height of 313 ft and contains approximately 200 000 cu yd of concrete. A general view of the dam and power house is shown by Fig. 2.

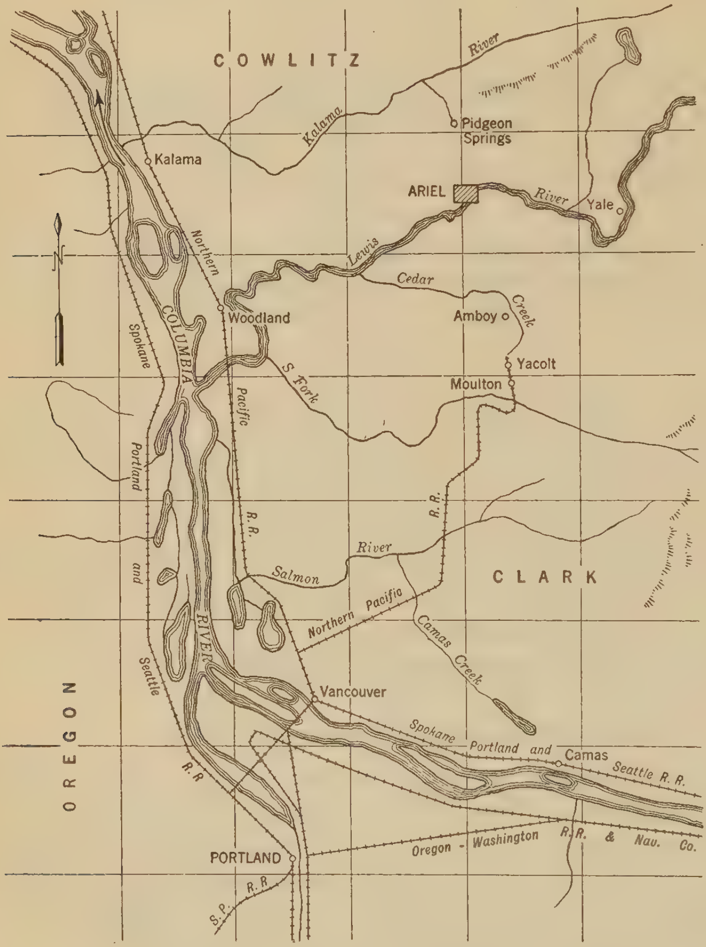

COWLITZ
River
Kalama
Pidgeon Springs
Kalama
ARIEL
River
Yale
Northern
Lewis
Cedar
Woodland
Amboy
Creek
COLUMBIA
Spokane
S. Fork
Yacolt
Moulton
Pacific
Portland
R. R.
R. R.
River
Salmon
CLARK
and
Northern Pacific
RIVER
Camas
Creek
Seattle
Vancouver
OREGON
Spokane Portland and Seattle R. R.
Camas
R. R.
Oregon - Washington R. R. & Nav. Co.
PORTLAND
R. R.
S. P.

FIG. 1.—LOCATION OF ARIEL HYDRO-ELECTRIC DEVELOPMENT.

A comparatively brief description of the development is given to provide a background for the proper understanding of the more detailed data which are presented later. The method of designing the arch is treated somewhat to show how the quantity of cement used governed the setting heat of the concrete and how this, together with the fast construction schedule, influenced the construction methods adopted. Consideration is also given to the pro-

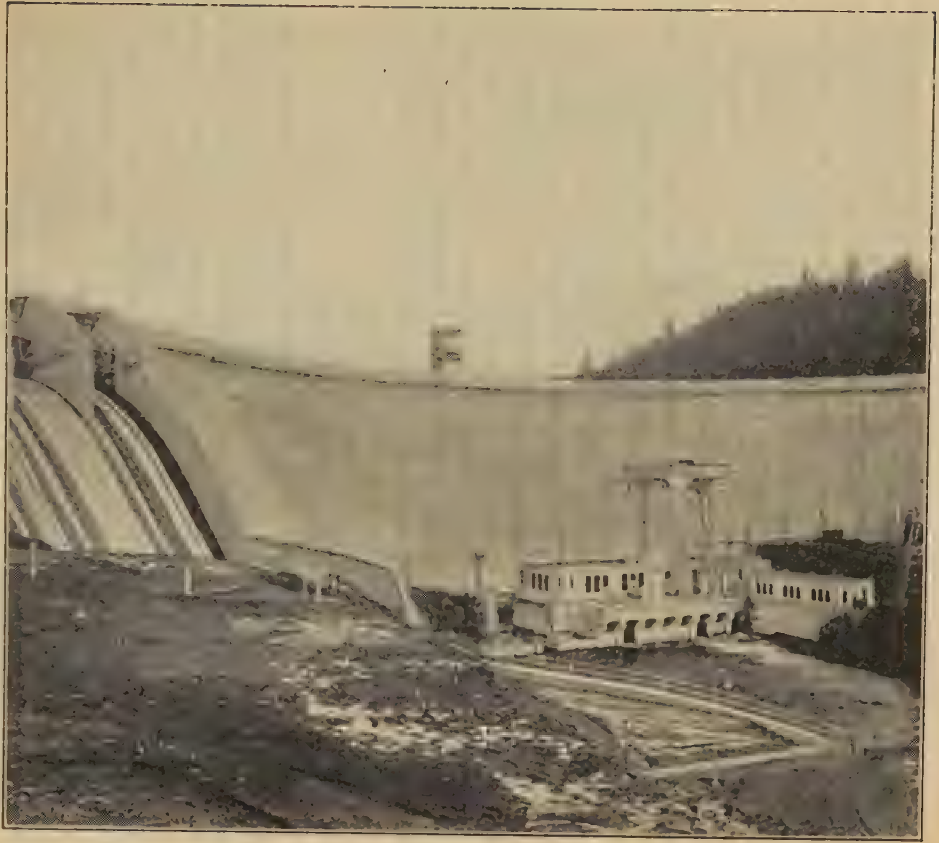


FIG. 2.—GENERAL VIEW OF DAM AND POWER HOUSE.

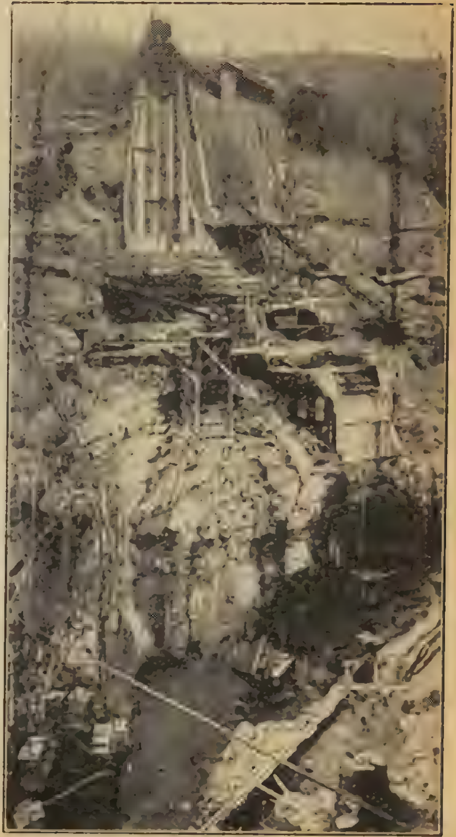


FIG. 3.—EXCAVATION OF DEEP CUT IN RIVER.

portioning of the concrete and to the special tests made to determine the strength of concrete taken from the dam itself by means of 4-in. diamond-drill cores.

The principal emphasis of the paper, however, is given to (1) a consideration of the amount of heat generated by the setting concrete and to the methods used to help dissipate this heat in some of the more rapidly constructed blocks; and (2) the deflection of the arch as measured at twenty-one different points. These measured deflections are compared with the computed deflections.

Investigations and Studies

Previous to starting the final layout of the dam and power house a large amount of diamond drilling was done in the river bed and on either bank to determine the character and tightness of the rock and the depth of the rock below the water surface in the river. The greater part of the drilling was done on lines at right angles to the river and about 100 ft apart. With the beginning of the detailed location and design of the dam and power house further drilling was done in the river bed and on either bank to determine the exact shape of the rock under the structure. Close determination of the depth, width, and shape of the river bed was essential on account of the necessity of unwatering the river, removing 190 000 cu yd of sand and gravel, preparing the foundations, and building the concrete dam up to a point above normal tail-water in the low-water period of four months between June 15 and October 15, 1930. Actual unwatering and excavation of this deep hole showed that the shape and depth determined by the drilling were extremely close. Fig. 3 shows this deep hole unwatered and being excavated.

Beyond the extreme north end of the dam it was found that the rock dropped about 100 ft below the surface. This low area is believed to have been a part of the bed of the Lewis River during a former glacial period. It is now filled with sand, gravel, boulders, and clay. The location of bed-rock in this section was determined partly by diamond drilling and partly by electrical prospecting methods. Further investigation of the area by pits and tunnels, and tests of the material for water-tightness, indicated that it would make a satisfactory dam. A low dike across this area was sufficient to provide ample free-board above reservoir level.

Plan of Development

While the site was not entirely ideal for an arch dam, because of its irregular shape and the low elevation of the rock on the north bank, it was sufficiently so to justify an arch across the river on the basis of quantity of material and cost. The exact location and layout of the different parts of the dam, including the arch, was decided upon after a considerable number of comparative studies had been made and the diamond drilling had indicated the position selected to be the most economical one. The presence of solid rock near the surface on the north bank of the river suggested the spillway layout shown by Fig. 4. The exact location, shape, and layout of the spillway were determined by very complete tests of a model in the hydraulic

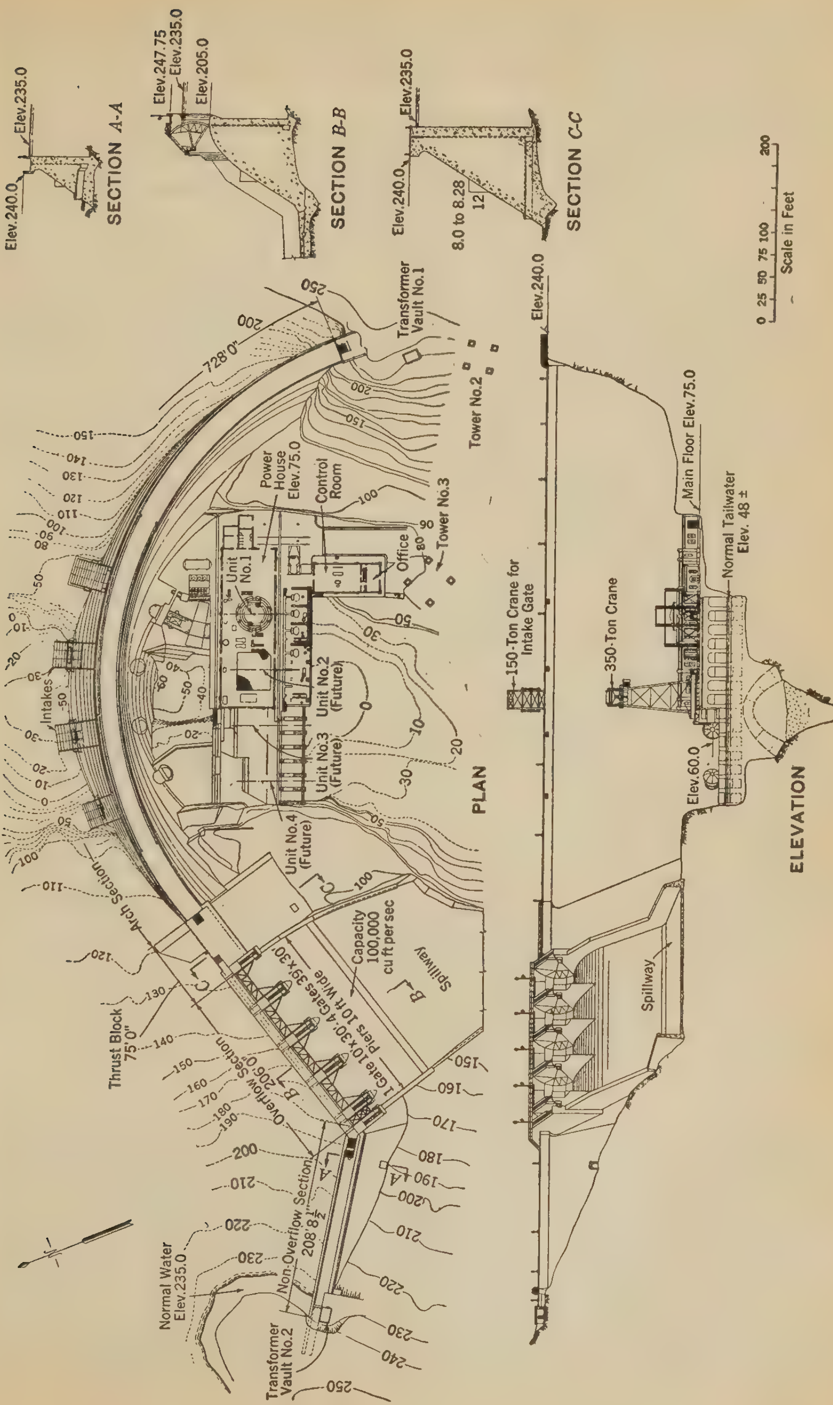

SECTION A-A
Elev.235.0
Elev.240.0
SECTION B-B
Elev.247.75
Elev.235.0
Elev.205.0
SECTION C-C
Elev.235.0
Elev.240.0
8.0 to 8.28
12
0 25 50 75 100 200
Scale in Feet
PLAN
728'0"
Transformer Vault No.1
Tower No.2
Tower No.3
Power House Elev.75.0
Control Room
Office
Unit No.1
Unit No.2 (Future)
Unit No.3 (Future)
Unit No.4 (Future)
Intakes
Arch Section
Thrust Block 75'0"
Overflow Section 206'0"
Non-Overflow Section 208'8½"
1 Gate 10'x30'; 4 Gates 39'x30'; Piers 10 ft Wide
Capacity 100,000 cu ft per sec
Spillway
Normal Water Elev.235.0
Transformer Vault No.2
ELEVATION
Elev.240.0
150-Ton Crane for Intake Gate
350-Ton Crane
Main Floor Elev.75.0
Normal Tailwater Elev. 48 ±
Elev.60.0
Spillway
A
B
C

FIG. 4.—PLAN AND ELEVATION OF DAM AND POWER HOUSE.

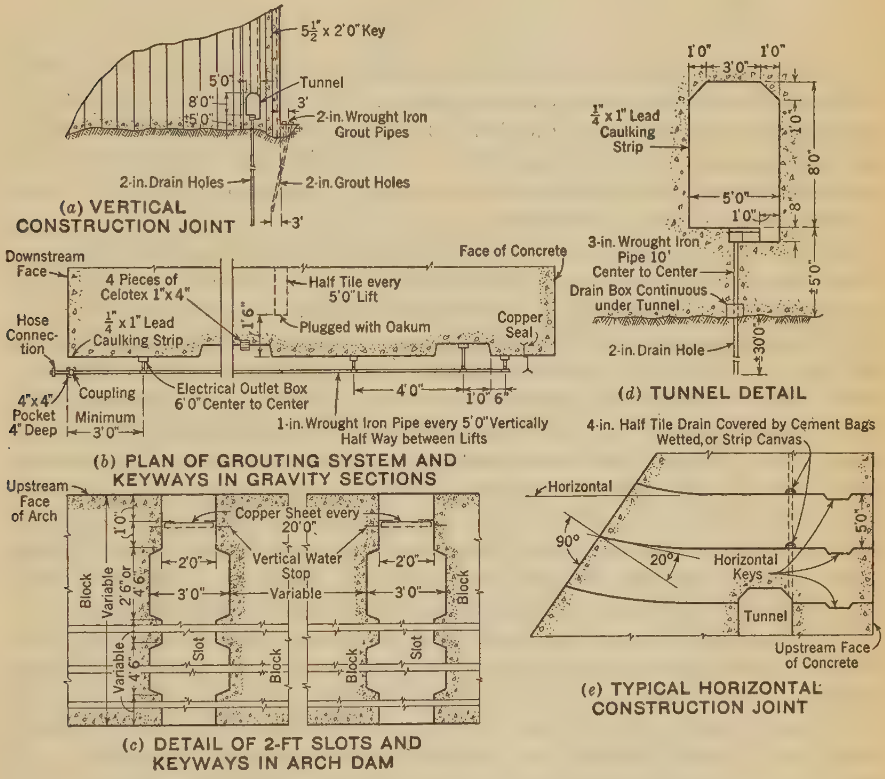
laboratory of the Carnegie Institute of Technology, Pittsburgh, Pa. The layout adopted permits discharging the water into the river under the bridge in a manner that allows a good "get-away" and a minimum interference with the bridge abutments.

The solid rock on the north bank of the river extended to a height sufficient to support only a part of the north end of the arch. The top 140 ft of the arch, therefore, is supported by a thrust-block, 75 ft long, this, in turn, being backed up by the 211 ft of massive spillway section of dam as shown by Fig. 4. The thrust-block itself is designed to take the entire thrust, shear, and twist of the arch above the rock. It is a massive section built in two parts with the construction joint grouted just previous to filling the reservoir. The overflow section of dam beyond the thrust-block is of the usual ogee type with piers that support five Taintor gates, 30 ft high. Four of the gates are 39 ft long, with the fifth one 10 ft long, the latter to be used for minor regulation of flow. The overflow dam was built in sections with all the joints grouted, as in the case of the thrust-block. The non-overflow section of the dam was built at an angle to the remainder so as to fit the rock to best advantage. This section was not grouted, but it, as well as the overflow and thrust-block sections, was provided with copper seals at the up-stream side of the vertical construction joints and with sheet metal cut-offs at the horizontal days' work joints. The percolation of water through the concrete was further prevented by providing 4-in. square, vertical holes through the concrete at intervals near the up-stream face. These holes connect with the top of the inspection gallery and were made by building 4-in. squares of Celotex into the concrete as it was carried up and then wetting and removing the Celotex to form the openings. Horizontal rows of half tiles between the vertical openings were placed on each day's work joint to catch any leakage and take it to the vertical holes.

Details of the grouting, keyways, joints, and seals, for both the gravity and arch sections, are shown by Fig. 5. The drain holes in Fig. 5(*a*) are 30 ft deep and are spaced on 20-ft centers in such a way as to alternate with the grout holes. The latter were drilled from 25 to 50 ft and were spaced on 10-ft centers, slanted inward at the bottom as shown. In Fig. 5(*e*), it is to be noted that the bedding planes are horizontal from the up-stream face to the half-way point, where they begin gradually to slope upward, to the maximum shown at the down-stream face.

Design of Arch Dam

The arch dam was designed by the "trial-load" method which assumes the total water load to be divided between the cantilever and arch elements in such a way that the deflections of the two systems of elements are equal at all points. This method of design involves considerable work, but it allows the stresses to be determined at as many points in the dam as is desired, this number being limited by the time and money allowed. This method has been used by the United States Bureau of Reclamation in designing and checking several important arched gravity dams. It permits making allowance for foundation and abutment deflections, for radial sides of cantilever elements, for loads carried by tangential shear and twist, seasonal variation in concrete temperatures, up-stream deflection of the dam when the reservoir is empty, and other miscellaneous factors, as well as moment, transverse shear, and thrust in both the arch and cantilever elements, which are usually considered. No allowance was made for the effect of water-soaking of the concrete at the up-stream face of the dam, for Poisson's ratio, or for probable reduction in maximum stresses due to the flow of concrete.


5½" x 2'0" Key
Tunnel
5'0"
8'0"
5'0"
3'
2-in. Wrought Iron Grout Pipes
2-in. Drain Holes
2-in. Grout Holes
3'
1'0"
1'0"
3'0"
¼" x 1" Lead Caulking Strip
1'0"
8'0"
5'0"
1'0"
8'
3-in. Wrought Iron Pipe 10' Center to Center
Drain Box Continuous under Tunnel
15'0"
2-in. Drain Hole
±30'0"
Downstream Face
4 Pieces of Celotex 1"x 4"
Hose Connection
¼" x 1" Lead Caulking Strip
11'6"
Half Tile every 5'0" Lift
Plugged with Oakum
Face of Concrete
Copper Seal
4"x 4" Pocket 4" Deep
Coupling
Minimum 3'0"
Electrical Outlet Box 6'0" Center to Center
4'0"
1'0"
6"
1-in. Wrought Iron Pipe every 5'0" Vertically Half Way between Lifts
4-in. Half Tile Drain Covered by Cement Bags Wetted, or Strip Canvas
Upstream Face of Arch
1'0"
Copper Sheet every 20'0"
Vertical Water Stop
2'0"
2'0"
Block
Variable
2'6" or 4'6"
3'0"
Variable
3'0"
Block
Slot
Block
Variable
4'6"
Block
Slot
Horizontal
90°
20°
Horizontal Keys
5'0"
Tunnel
Upstream Face of Concrete

(a) VERTICAL CONSTRUCTION JOINT

(b) PLAN OF GROUTING SYSTEM AND KEYWAYS IN GRAVITY SECTIONS

(c) DETAIL OF 2-FT SLOTS AND KEYWAYS IN ARCH DAM

(d) TUNNEL DETAIL

(e) TYPICAL HORIZONTAL CONSTRUCTION JOINT

FIG. 5.—CONSTRUCTION JOINT AND GROUTING DETAILS.

The following assumptions were made: (1) Weight of concrete, 145 lb per cu ft; (2) weight of water, 62.5 lb per cu ft; (3) maximum elevation of reservoir, 240 ft; (4) minimum tail-water elevation, 50 ft; (5) modulus of elasticity of rock, 4 000 000 lb per sq in.; (6) modulus of elasticity of concrete, 2 000 000 lb per sq in.; (7) modulus of elasticity of concrete in shear, 96 000 000 lb per sq ft; (8) maximum allowable vertical or horizontal compressive stress of concrete, 600 lb per sq in.; (9) maximum allowable tensile stress of concrete in arch elements, 100 lb per sq in.; (10) the uplift pressure at the base of the arch section varies from full reservoir pressure at the up-stream side to zero or tail-water pressure, at the down-stream side (these pressures to be applied to one-half the horizontal area of the base); (11)

cement content, 1 bbl per cu yd; and (12) temperature drop in arch concrete after the slots have been closed must not exceed the following:

Elevation	Temperature drop, in degrees Fahrenheit
220	11
180	9
120	5
60	3
− 20	2.5

Assumption (7) was adopted to compensate for the non-linear distribution of shear, calculations being made on the basis of straight-line distribution throughout the thickness of the dam. Referring to Assumption (9), cracking was assumed to take place wherever the existence of tension in the cantilever elements was indicated by the computations, so that the entire load is resisted by compressive stresses.

One of the governing factors in the design of the arch dam was the comparatively short construction period adopted. With the first work of camp and road construction started on November 1, 1929, the first concrete was placed in the gravity section of the dam on April 22, 1930, and in the arch on June 11, 1930. The river was diverted through the tunnel on June 25, 1930. The arch dam was complete, including slot-filling, on April 18, 1931. Approximately 200 000 cu yd of concrete were placed in the arch during this time. This, in itself, was not an unusually difficult schedule. The limiting factor was the necessity of unwatering the river, excavating for a depth of about 125 ft below natural river surface, preparing the foundation, and building the concrete arch up about 135 ft in the period between June 25 and about October 15, when the fall floods could be expected. Failure to meet this requirement might have delayed the operation of the plant a year.

The first concrete was placed on the river bottom on September 13, 1930, and continued on a 24-hour basis until October 1 when Elevation 60 was reached. The largest volume of concrete placed in any one day was 2 234 cu yd on September 25. During the month of October 54 460 cu yd of concrete were placed. During the months of July, August, September, and October, the total was 181 000 cu yd. It will be seen, therefore, that although the schedule as a whole was not unusually fast more than one-half the total concrete in the dam and power house was placed during a 4-month period.

At the beginning of the design work information was secured as to the quantity of cement used in several private and Government gravity and arch dams and the resulting temperatures. Difficulty was soon encountered in determining an adequate design for the arch on account of the limited time available to dissipate the setting heat of the concrete. It was found that thickening the cross-section to reduce the load stresses increased: (1) The setting heat; (2) the temperature drop to be taken care of in the design; and (3), the temperature stresses produced in the arch. The increases in tensile stress produced at the extrados at the abutments and at the intrados at the crown, due to the increased temperature drop, were especially undesirable. Finally, it was found necessary to adopt the block construction instead of the grouted-joint construction in order to dissipate the setting heat in the limited time available. The allowance for the temperature drop with this method of construction is indicated in the tabulation under Assumption (12). Division into comparatively short blocks, with 2-ft slots between (see Figs. 3 and 6), permitted the development of an economical and rational design which will not be subject to unreasonably high secondary stresses.

The arch is divided into blocks approximately 30 ft long (see Fig. 6), except in that portion behind the power house where the penstocks come through, where they have a length of about 40 ft at the up-stream face of the dam. The openings, or slots, between blocks, were kept as short as possible for two reasons: First, to keep to a minimum the quantity of concrete that would have to be placed rapidly at the very end of the job; and, second, to minimize the heat generation that would come from this volume of new concrete. The 2-ft slots adopted were just sufficient to allow proper handling of the end forms and also to allow for the proper placing of the final slot concrete.

The stresses in the arch rings were computed at forty-five different points at five different elevations. At each elevation the stresses were determined at the crown and abutments and at three points between. The stresses finally computed in the arch rings, adjusted for the effects of tangential shear and twist, are shown by Table 1. The stresses finally computed in the cantilever elements, adjusted for the effects of tangential shear and twist, are shown by Table 2. The stresses are shown for both the up-stream and down-stream faces of the dam in both the vertical and inclined directions. In several cases the cantilever section is indicated as having a crack extending a considerable distance in from the up-stream face. In those cases the compression is computed as zero at the end of the crack, the total compression being taken by the remainder of the section. It will be seen from Tables 1 and 2 that the maximum stress in the arches is considerably less than 600 lb per sq in. in every case, with one point showing a maximum of 560 lb per sq in. The maximum stresses in the dam occur in the cantilever elements, the 600-lb limit being exceeded slightly in two instances.

Construction Details of Arch Dam

The arch dam extends between the rock abutments on either side of the river with the up-stream radius varying from 397 ft at the top to 247 ft at Elevation 40. The angle between the ends of the arches varies between 108° at the top and 80° at normal tail-water at Elevation 50. The plan and maximum section of the arch are shown in Figs. 6 and 7. It will be noted that the maximum length of the arch at the top is 728 ft, the maximum height, 313.4 ft, and the maximum thickness, 93 ft. Although the arch is vertical at the up-stream side at its deepest part it does overhang about 5 ft near the abutments. Fig. 6(*a*) also shows the four main penstocks, the house unit penstock, and the location of the various slots. The lines shown for the slots indicate their intersection with the outside faces of the concrete. In every case the slots extended radially through the dam at each 10-ft elevation.

TABLE 1.—Stresses in Arch Elements, in Pounds per Square Inch

(Reservoir elevation, 240; + = compression; — = tension)

Elevation	North Abutment		Three-Fourth Point		Half-Point		Quarter-Point		Crown		Quarter-Point		Half-Point		Three-Fourth Point		South Abutment	
	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos	Intra-dos	Extra-dos
220....	+166	+233	+314	+87	+87	+294	—28	+417	+81	+312	+255	+140	+349	+48	+267	+129	+171	+220
180....	+560	—107	+325	+122	+49	+392	—21	+46	+145	+298	+286	+160	+348	+99	+294	+153	+72	+370
120....	+372	—105	+182	+76	+33	+221	—33	+284	+31	+223	+142	+116	+216	+44	+201	+59	+64	+192
60....	+402	—16	+408	—14	+299	+131	+117	+335	+21	+441	+117	+335	+299	+131	+408	—14	+402	—16
—20....	+402	—43	+292	+77	+185	+203	+95	+323	+61	+369	+95	+323	+185	+203	+292	+77	+402	—43

TABLE 2.—Stresses in Cantilever Element, in Pounds per Square Inch (See Fig. 6(*b*))*

Cantilevers	*A*		*C*		*E*		*G*		*I*		*K*		*M*	
Direction of stress	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical	Inclined	Vertical
							Up-Stream Face							
Elevations:														
220..................	+36	+36	+41	+41	+42	+42	(9)	(9)	+37	+37	+34	+34	+39	+39
180..................	+73	+73	+95	+94	+111	+110	+94	+93	+75	+74	+56	+56	+80	+80
140..................	+10	+10	+49	+38	+68	+66	+21	+20	+ 5	5	(9)	(9)	+20	+20
120..................	(13)	(13)	(4)	(4)	+25	+25	(10.5)	(10.5)	(16)	(16)	(21)	(21)	(11)	(11)
110..................	....	....	....	....	....	....	....	....	....	....	....	....	(30)	(30)
100..................	(17)	(17)	(16)	(16)	(11)	(11)	(23)	(23)	(22)	(22)	(33)	(33)	....	....
90..................	(34)	(34)	....	....	....	....	....	....	....	....	....	....	....	....
75..................	....	....	....	....	....	....	....	....	(45)	(45)	....	....	....	....
60..................	....	....	(27)	(27)	(28)	(28)	(23.5)	(23.5)	....	....	....	....	....	....
20..................	....	....	(35)	(35)	(33)	(33)	(36)	(36)	....	....	....	....	....	....
0..................	....	....	(56)	(56)	....	....	(55.5)	(55.5)	....	....	....	....	....	....
—20..................	....	....	....	....	(31)	(31)	....	....	....	....	....	....	....	....
—50..................	....	....	....	....	(42)	(42)	....	....	....	....	....	....	....	....
							Down-Stream Face							
220..................	+37	+37	+41	+41	+43	+43	+41	+41	+37	+37	+33	+33	+39	+39
180..................	+71	+71	+95	+94	+113	+112	+99	+98	+74	+73	+55	+55	+80	+80
140..................	+151	+144	+111	+108	+94	+93	+147	+145	+162	+159	+177	+172	+136	+127
120..................	+250	+238	+197	+193	+174	+172	+250	+247	+295	+292	+333	+323	+230	+211
110..................	....	....	....	....	....	....	....	....	....	....	....	....	+451	....
100..................	+298	+248	+303	+291	+272	+269	+414	+409	+396	+341	+540	+337	....	+414
90..................	+440	....	....	....	....	....	....	....	....	....	....	....	....	....
75..................	....	....	....	....	....	....	....	....	+808	....	....	....	....	....
60..................	....	+324	+441	+324	+438	+421	+511	+297	....	+647	....	....	....	....
20..................	....	....	+450	+432	+533	+512	+470	+452	....	....	....	....	....	....
0..................	....	....	+647	....	....	....	....	....	....	....	....	....	....	....
—20..................	....	....	....	+622	+571	+549	+632	+607	....	....	....	....	....	....
—50..................	....	....	....	....	+584	+561	....	....	....	....	....	....	....	....

* Numerals in parentheses denote points at which crack occurred and at which stress is zero; thus, (13) denotes a crack extending 13 ft from the face.

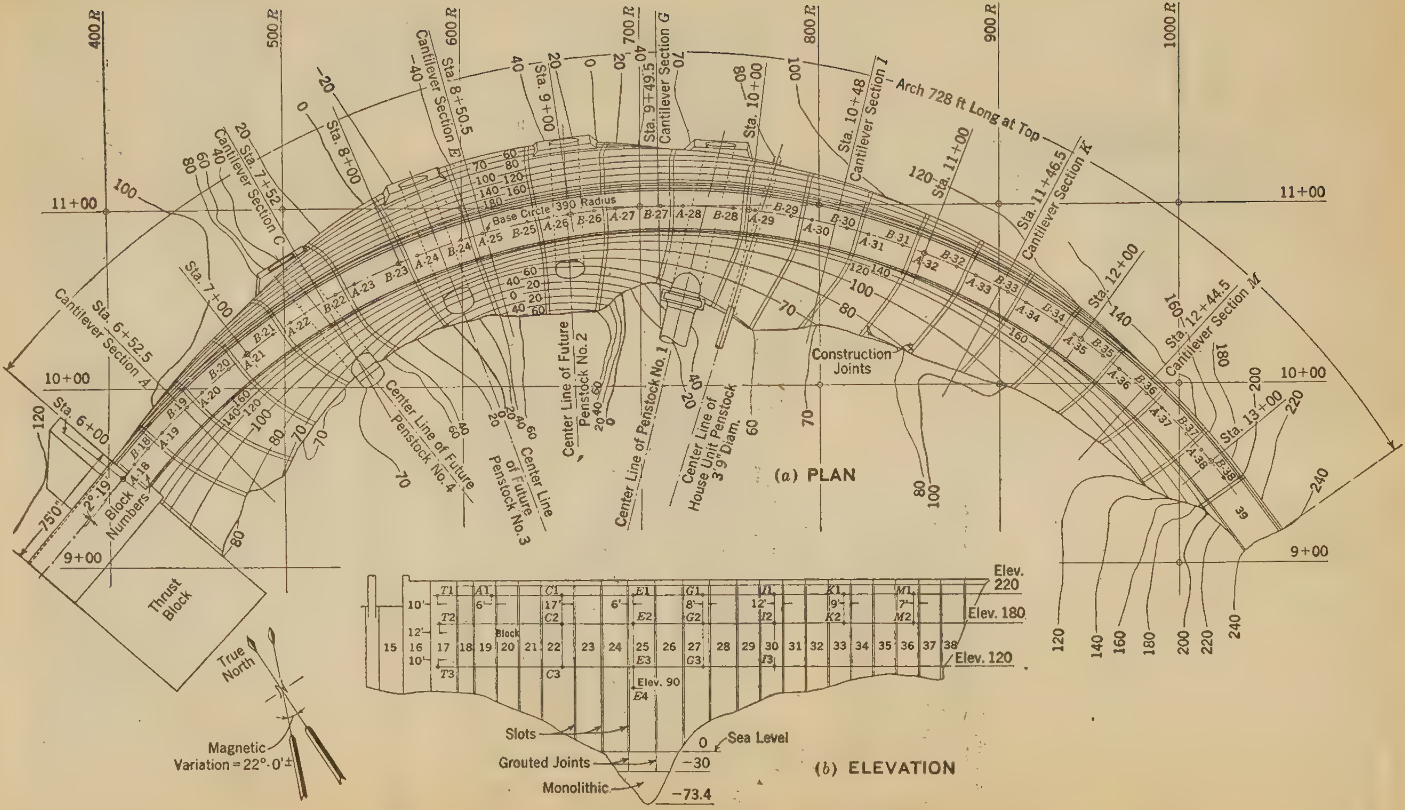

(a) PLAN
Arch 728 ft Long at Top
Base Circle 390 Radius
Construction Joints
Block Numbers
Thrust Block
True North
Magnetic Variation = 22°-0'±
Sta. 6+52.5 Cantilever Section A
Sta. 7+52 Cantilever Section C
Sta. 8+50.5 Cantilever Section E
Sta. 9+49.5 Cantilever Section G
Sta. 10+48 Cantilever Section I
Sta. 11+46.5 Cantilever Section K
Sta. 12+44.5 Cantilever Section M
Center Line of Future Penstock No. 4
Center Line of Future Penstock No. 3
Center Line of Future Penstock No. 2
Center Line of Penstock No. 1
Center Line of House Unit Penstock 3'9" Diam.
400 R
500 R
600 R
700 R
800 R
900 R
1000 R
(b) ELEVATION
Elev. 220
Elev. 180
Elev. 120
Elev. 90
Sea Level
Slots
Grouted Joints
Monolithic
−30
−73.4

FIG. 6.—BLOCK NUMBERING AND LOCATION OF DEFLECTION POINTS.

This resulted in curiously warped openings in which the concrete sometimes overhung for several feet. The end of each block was built with approximately half its area depressed for a depth of 6 in. by means of wooden forms 4 ft 6 in. long. These depressions provided a good bond between the old

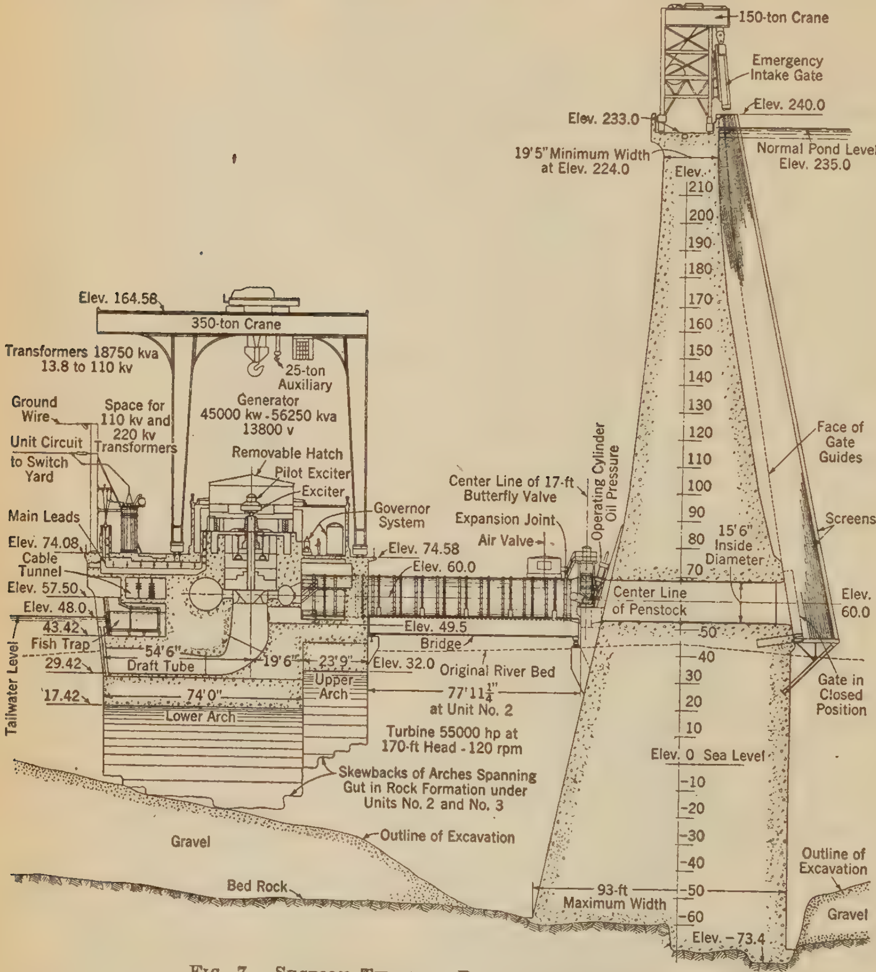

150-ton Crane
Emergency Intake Gate
Elev. 240.0
Elev. 233.0
Normal Pond Level Elev. 235.0
19'5" Minimum Width at Elev. 224.0
Elev. 210
200
190
180
170
160
150
140
130
120
110
100
90
80
70
Elev. 164.58
350-ton Crane
Transformers 18750 kva 13.8 to 110 kv
25-ton Auxiliary
Generator 45000 kw - 56250 kva 13800 v
Ground Wire
Space for 110 kv and 220 kv Transformers
Unit Circuit to Switch Yard
Removable Hatch
Pilot Exciter
Exciter
Main Leads
Governor System
Center Line of 17-ft Butterfly Valve
Expansion Joint
Air Valve
Operating Cylinder Oil Pressure
Face of Gate Guides
Screens
15'6" Inside Diameter
Elev. 74.08
Cable Tunnel
Elev. 57.50
Elev. 48.0
43.42
Fish Trap
29.42
17.42
Tailwater Level
Elev. 74.58
Elev. 60.0
Center Line of Penstock
Elev. 60.0
Elev. 49.5
Bridge
54'6"
Draft Tube
19'6"
23'9"
Upper Arch
74'0"
Lower Arch
Elev. 32.0
Original River Bed
77'11¼"
at Unit No. 2
Turbine 55000 hp at 170-ft Head - 120 rpm
50
40
30
20
10
Gate in Closed Position
Elev. 0 Sea Level
-10
-20
-30
-40
-50
-60
Skewbacks of Arches Spanning Gut in Rock Formation under Units No. 2 and No. 3
Gravel
Outline of Excavation
Outline of Excavation
Bed Rock
93-ft Maximum Width
Gravel
Elev. -73.4

FIG. 7.—SECTION THROUGH DAM AND POWER HOUSE.

and the new concrete when the slots were filled, and minimized the possibility of leakage through the joints. No vertical seals were used in these joints as it was believed that the corrugated construction and the fact that the arch would be under compression would allow little if any leakage. The details of these keyways are shown by Fig. 5.

From the extreme bottom part of the arch at Elevation − 73.4 to Elevation − 30, the concrete was placed monolithically because of the comparative narrowness of the canyon. From Elevation − 30 to Elevation 0 the concrete was built in three blocks with vertical construction joints between. These joints were provided with a piping system for grouting, the piping being placed horizontally at each 5-ft elevation and opening into electrical conduit boxes placed against the old concrete at intervals of 4 ft 6 in. on the face of each block. Above Elevation 0 the arch was built in blocks with the 2-ft slots between, except for Blocks 20, 21, and 22 (Fig. 6(*b*)), which were left low through the winter of 1930-31 for the purpose of passing excess flood-waters that could not be handled by the diversion tunnel. These blocks were finally built with only construction joints between them. These joints also were provided with a piping system for grouting after they had been cooled.

The foundation of the arch was thoroughly grouted at the up-stream face by means of diamond drill holes extending 20 to 50 ft into the rock on 10-ft centers. This grouting, as well as the grouting of the joints, noted previously, was done just before filling the reservoir, which began on May 13, 1931. The top of the arch on the down-stream side has a simple curved cornice for architectural effect. In the top 7 ft considerable concrete was saved by building only comparatively thin walls at the up-stream and down-stream sides. The wall at the up-stream side is provided with copper seals at each joint, while the down-stream wall has asphalt strips in the construction joints to allow for possible expansion and contraction.

Construction Methods and Materials

For the purpose of building the arch dam and power house, the river bed was unwatered by means of rock-filled timber cribs built on the river gravel above and below these structures. The total volume of material removed was 190 000 cu yd, 40 000 cu yd of this being taken out before the river was unwatered, by means of derricks on the river banks. Some of the gravel removed from the river bed at the dam was used for concrete aggregate. The main gravel deposit in the bar below the lower coffer-dam was removed from the river by means of buckets on two slack-line cableways covering the entire area involved. These cableways dumped into a hopper on the north river bank. The gravel was carried by a belt conveyor to the screening and washing plant where it was washed and graded into three sizes and stored in piles.

All cement was delivered in bulk to the contractor's siding twelve miles from the plant. It was hauled to the plant by trucks having specially constructed 50-bbl bodies provided with compressed-air jets for loosening the cement during unloading. It was found, however, that the use of air was unnecessary. At the job these trucks entered a cement shed and dumped the cement directly into storage bins. From the storage bins, the cement was taken to the top of the concrete mixing plant by a belt conveyor.

The sand and the two sizes of concrete aggregate were taken from the stock piles to the top of the concrete mixing plant separately by a belt conveyor. Each size of aggregate and the cement were accurately weighed separately and then discharged into 2-yd mixers for 2½ min of mixing. The concrete was discharged into 2-yd bottom-dump buckets on small industrial cars. The cars were hauled from the mixer to the placing point at the dam by gasoline locomotives. The buckets were handled by derricks to some extent, but mostly by "whirleys." Each block of the arch was built up with the whirley located on an adjacent block. The work was carried on from each end and at two levels, the whirleys moving on to the newly completed block about two days after its completion. The whirleys and derricks spotted the concrete buckets accurately over the place where the concrete was finally to rest. Almost no rehandling of the concrete was necessary or allowed.

The arch blocks were built with unlined wooden forms, 10 ft. high by 12 ft long. This length was short enough so that the chords closely approached the circular arcs on which the dam was laid out. The forms were placed in proper position by measurements from a base circle on which there were two transit points in each block. Six angle and distance measurements from each of the two points were sufficient to determine the location of the forms accurately. The necessary angles and distances for each 10-ft elevation were prepared in tabular form, the distances being scaled from a large scale plan of the dam and the angles computed therefrom. Experience in the design of a previous arch dam in which all distances and angles were calculated and then checked by measurement from the drawing indicated that the scaling of the distances from the drawing and the calculation of the angles were sufficiently accurate.

CONCRETE PROPORTIONING AND STRENGTH

The sand and coarse aggregate were secured partly from the excavation in the river bed at the dam, but mostly from a gravel bar about $\frac{1}{4}$ mile down stream. The aggregate consisted of well rounded particles of andesite and basalt lava, most of the rock being dense and heavy, but some of it being somewhat vesicular. The vesicular rock and pumice were largely removed in the washing and screening process. The specific gravity of the sand averaged 2.47 and that of the coarse aggregate, 2.38. The aggregate as a whole had an absorption value of 3.9% in 24 hr. With the exception of being slightly light in weight the aggregate was sound and entirely suitable for concrete. The weight of saturated concrete in place in the dam, as determined from a considerable number of samples removed after some months, was about 147.5 lb per cu ft

The concrete in the dam had a nominal mix of 1:2$\frac{1}{2}$:5. To attain maximum density, it was proportioned by weight, as follows:

Mix by volume	Material weights for 2-yd batch
1 part cement	Cement, 752 lb
2.4 parts sand, 0 in. to $\frac{5}{16}$ in..........	Sand, 1 900 lb
2.0 parts pebbles, $\frac{5}{16}$ in. to 1$\frac{1}{2}$ in........	Pebbles, 1 450 lb
4.6 parts cobbles, 1$\frac{1}{2}$ in. to 4 in.........	Cobbles, 3 450 lb
Water, 64 gal.	

The materials used for making the concrete, and the concrete itself, were continuously inspected by a special force of four men. The inspectors had complete control of the concrete mixture and its handling and placing in the dam. In addition, they were responsible for the proper preparation of the concrete surfaces on which new concrete was to be placed and for the proper handling of the seals, cut-offs, drains, etc. The testing of the cement at the mills and at the laboratory, and the testing of the concrete materials and the concrete itself were done in accordance with A. S. T. M. standard methods in a modest but well equipped testing laboratory at the dam. A total of 379 831 bbl of cement were delivered to the job in bulk and its acceptable quality was determined by a total of 1 969 tests. The physical characteristics of the aggregate and the concrete made from it are given in Table 3. These data are typical of those attained throughout the job.

TABLE 3.—Typical Test Report on Concrete and Aggregate

Date September 30, 1930
Serial No. 127 Location Block 24—Elev. 40–50.
Temperature Max 74° F. Min 41° F. Test represents 1506 Cu. Yds. Concrete

SIEVE ANALYSIS
AGGREGATE

Sand				Fine Gravel				Coarse Gravel			
Sieve No.	Weight	Percent.		Sieve No.	Weight	Percent.		Sieve No.	Weight	Percent.	
		Re-tained	Greater			Re-tained	Greater			Re-tained	Greater
4	....	.....	.....	3"	....	.....	.....	3"	....	.....	......
8	50	10.0	10.0	2"	....	.....	.....	2"	....	.....	.....
14	105	21.0	31.0	1½"	....	.....	.....	1½"	23	70	70
28	179	35.8	66.8	1"	....	.....	.....	1"	....	.....	.....
48	118	23.6	90.4	¾"	12	35	35	¾"	9	27	97
100	39	7.8	98.2	⅜"	13	38	73	⅜"	1	3	100
—100	9	1.8	100.0	No. 4	9	27	100	No. 4	.....	.....	.....
Total	500	100.0	296.4	.....	34	100	208	......	33	100	267
Fineness Modulus....			2.96				7.08				867
Percent. of Mix......			27				22				51

Mixed Aggregate

Sieve No.	Sand	Fine Gravel	Coarse Gravel	Final
100.......	98	100	100	99
48.......	90	100	100	97
28.......	67	100	100	91
14.......	31	100	100	82
8.......	10	100	100	76
4.......	..	100	100	73
⅜".....	..	73	100	67
¾".....	..	35	97	58
1½"....	..	...	70	36
3"......	..	...	...	..
F. M.....	2.96	7.08	8.67	6.78

Sand Test

	Compressive	Tensile
Local....	lbs.	lbs.
7 day....	....	....
28 day...	....	....
Average..	....	....
Standard.	....	....
7 day....	....	....
28 day...	....	....
Average..	....	....

Miscellaneous Properties

Aggregate

	Sand	Fine Gravel	Coarse Gravel	Final
Voids..........	35.0	36.2	34.0	18.0
Wt lbs cu ft......	104	96	98	124
Spec. Gr.......	2.57	2.41	2.38	2.43
% Abs. 24 Hrs..	2.0	4.6	4.4	3.8
% Moisture....	5.9	4.1	1.3	3.2
% Loss by Dec.,	1.4	.....	.....	......
Soundness......	O.K.	O.K.	O.K.	......
Colm. No......	1	.....	.....	......

Concrete

Mix..................	1–2.4–2.0–4.6
Bbls Per Yard..........	1.0
w/c..................	1.05
Density...............	86.8
Relative Consistency....	1.1
Slump................	1"

Test

Comp. Str......	7 da	28 da	60 da
Lbs. per sq inch..	1 660	2 810	3 180
Comp. Str.......	3 mo	6 mo	....
Lbs. per sq inch .	3 530	3 750	....

Remarks:

Two hundred and fifty-three 6 by 12-in. test cylinders were made during the construction of the dam and tested, in the laboratory, for compressive strength at various ages. The average strength of the various cylinders at 7, 28, 60, 90, and 180 days is shown by Fig. 8, from which it will be seen that the average compressive strength of the concrete was about 2 800 lb per sq in., at 28 days, and 3 750 lb per sq in., at 180 days.

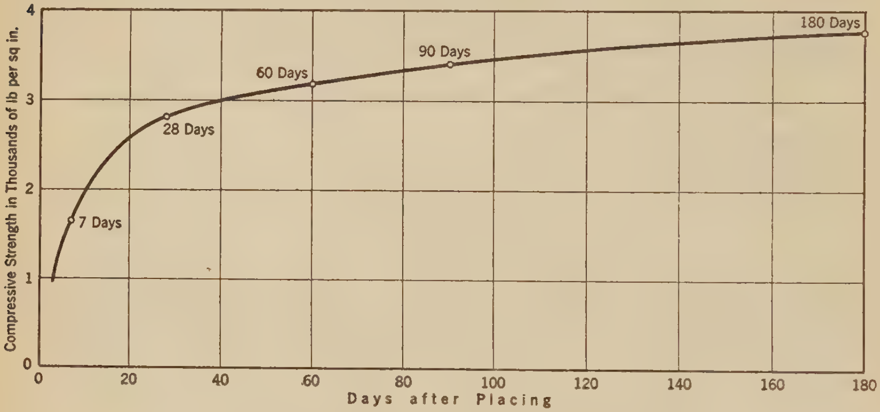


FIG. 8.—TEST CYLINDERS—AVERAGE COMPRESSIVE STRENGTH.

4-INCH DIAMOND DRILL CORES FROM INTERIOR OF DAM

For a long time there has been a question as to whether the results obtained from laboratory tests of concrete cylinders are representative of the strength of the concrete in place in a structure. To determine this point for the Ariel Dam six holes were drilled into it at different locations, angles, and depth by means of a diamond drill using a special barrel which recovered $3\frac{1}{8}$-in. cores, 5 ft long. A total drilling of 253 ft gave 243 ft 8 in. of core available for testing. This core was cut into 258 cylinders, 8 in. long, and the remainder was used for determining moisture content and weight. Some of these holes were drilled at an angle with the horizontal so as to cut

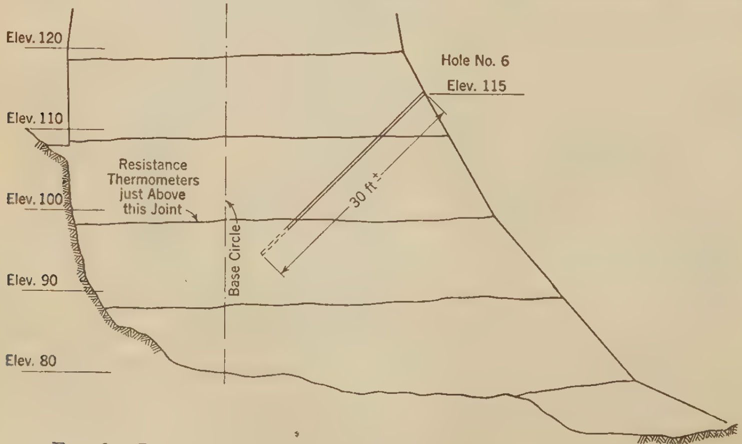


FIG. 9.—LOCATION OF DIAMOND DRILL TEST HOLE IN BLOCK 32.

across the days' work joints to determine the bonding of the old and new concrete at these points. The location in elevation of one of these six holes is shown by Fig. 9.

The high percentage of core recovery indicates the excellent quality of the concrete; many cores were obtained the full length of the core barrel (5 ft), and considerable difficulty was encountered at times in breaking off the core. In all the holes only one batch of honey-combed concrete, 8 in. thick, was encountered. An excellent bond existed between the various days' work joints. These joints could be identified in only a few places and then only when different brands of cement were used. Fig. 10 shows the weight and compressive strength of the cores taken from the thrust-block. On account of the presence of large cobbles in the cores the individual tests showed considerable variation in compressive strength; it is believed that larger cores would have given even higher strengths. A comparison of the time-strength relation of the standard 6 by 12-in. test cylinders and the 4 by 8-in. cores is shown by Fig. 11. The number of concrete cylinders tested at six months or earlier is shown, for each point on the curves, in Table 4. The core specimens

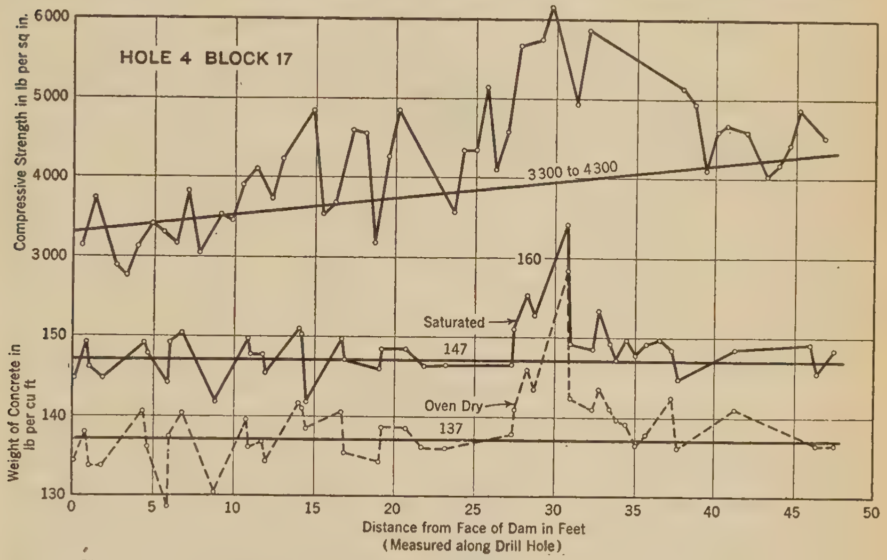


FIG. 10.—COMPRESSIVE STRENGTH AND WEIGHT OF DIAMOND DRILL CORES.

TABLE 4.—NUMBER OF CONCRETE CYLINDERS AVERAGED AT VARIOUS AGES (SEE FIG. 11)

Point No.	NUMBER OF CYLINDERS AVERAGED	
	Cement No. 1	Cement No. 2
1	60	193
2	60	193
3	60	193
4	14	8
5	14	3

were tested at ages greater than nine months. In Fig. 11 the number beside the triangle shows the core hole from which the point is taken. While the results indicate that the average strength of some of the cores is less than might be expected from the curve exterpolated beyond 180 days it is believed that these irregularities are explainable by the presence of the large cobbles.

The resistance thermometers embedded in the concrete, close to these holes, showed temperatures in the concrete around the holes as great as 61.9° C (144° F), and a temperature of 50° C (122° F) was common. To test the accuracy of the resistance thermometers, mercury thermometers were placed in the diamond drill holes in Blocks 17 and 32 (Fig. 9). A comparison of the readings of these thermometers showed a variation of 0.9° C to 3.0° C. This difference can be explained by the fact that the various resistance thermometers were from 4 to 15 ft away from the hole and as there was a marked temperature gradient in the concrete sloping toward the hole there would be a difference in the readings. Fig. 12 shows the temperatures in Hole 6, Block 32, at varying distances from the face of the concrete and for a period of several months from the time the concrete was placed. The curves were plotted from data read on thermometers embedded in the block as well as from thermometers inserted in the drill hole.

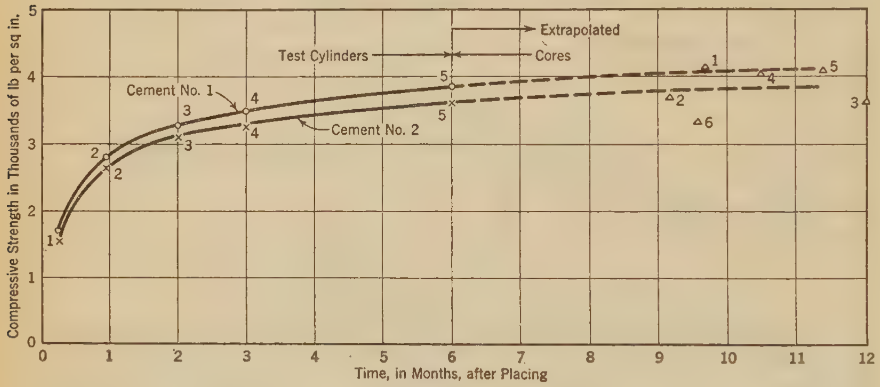


FIG. 11.—COMPARATIVE COMPRESSIVE STRENGTH OF TEST CYLINDERS AND CORES.

From Fig. 10 it is evident that the high temperatures in this dam (as great as 61.9° C) have caused no loss in the strength of the concrete subject to that temperature in the interior of the dam. On the contrary, it is evident that concrete 9 to 12 months old increased in strength from the surface inward and also from the foundations upward. At and near the foundations lower temperatures prevailed in the concrete due to the cooling effect of the rock. These conclusions seem to be well corroborated not only by Fig. 10, but by the results of the tests of the cores from all the holes.

In the process of obtaining the diamond drill cores they became so thoroughly wet that the moisture content of the concrete in place in the dam could not be determined from these core specimens with positive assurance.

However, an excellent opportunity to check the moisture content of the concrete was afforded in the summer of 1930 when a section of inspection tunnel 10 ft long was cut through old concrete near the base of the thrust-block by the plug-and-feather method. This concrete would not absorb additional moisture, while the loss of moisture upon oven-drying was the same as for the cores. Tests for moisture content, made on samples taken from other parts of the dam, showed the same saturated moisture condition. The concrete in this dam was saturated to within 2 or 3 ft of the exterior surfaces which had been exposed for 8 to 12 months.

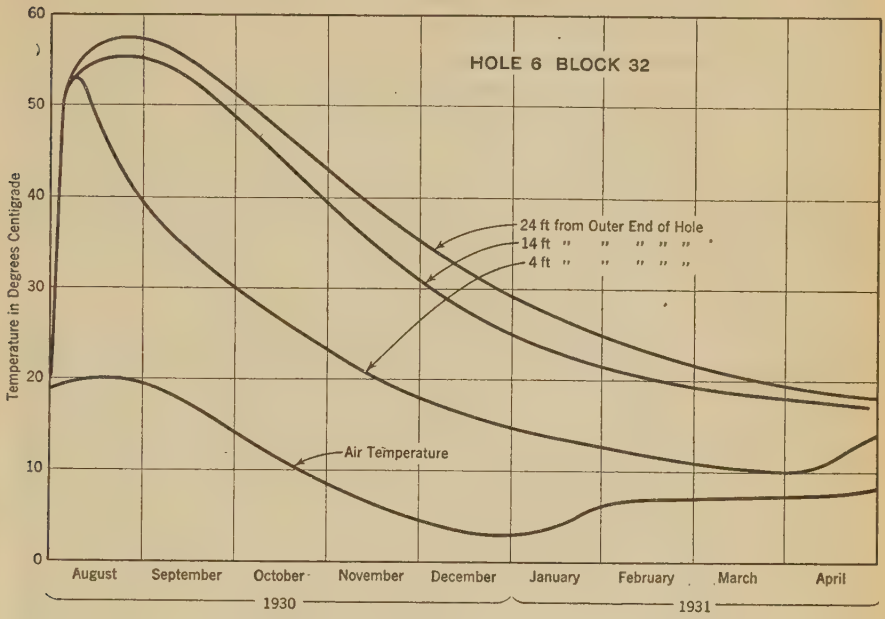


Fig. 12.—Temperatures in a Diamond Drill Hole.

The average weight of the concrete, as determined from the cores, was 147.5 lb per cu ft as it came from the dam saturated with water and 138 lb after being oven-dried. The question of the actual weight of concrete in place in dams needs further investigation; it has a marked effect on design and means a real saving or waste of money, because weight will materially influence the cross-section of a dam. In securing cores from concrete dams it is desirable that larger ones be obtained in order to eliminate, as far as possible, the effect of the large pieces of aggregate. If possible, 6-in. cores should be recovered to approximate the diameter of the standard test cylinders.

Heating of Concrete and Effect on Design of Dam

It is well known that a mass of concrete, in setting, produces considerable heat. Opinion as to the effect of high temperatures on the quality and strength of concrete is varied. One of the principal items governing the quantity of heat produced is the amount of cement used and the fineness to which it has been ground. In the case of the arch dam at Ariel the original specifications and designs called for 1.2 bbl of cement per yd. Test cylinders made from the cement and aggregate to be used in the arch indicated that 1 bbl of cement per yd would be sufficient. In view of the rapid construction schedule adopted the possibility of limiting the temperature rise by using cement ground coarser than usual was considered, but it was recognized that this would result in a loss of strength. The use of cement ground to the customary degree of fineness and limited to 1 bbl per yd was decided upon therefore as the most feasible scheme. Another feature governing the maximum temperature which the setting concrete may attain is the temperature at which the concrete mass itself is placed. A low placing temperature is desirable, but this cannot be controlled economically because it is governed to a considerable extent by the season.

Reference has been made to the allowance in the design for drop in temperature after the closing of the arch. Aside from any harmful effect that high temperatures might have on the quality and strength of the concrete it was desirable to limit the temperature rise in order to have the concrete cooled down to the ultimate annual mean temperature as quickly as possible so that the arch could be closed at an early date. It was evident during the design period that the maximum temperatures could not be brought down to the proper point for closing on schedule if the arch were built in the usual manner with only construction joints between adjacent blocks of concrete. Consequently, a method was adopted whereby it was built in blocks about 30 ft long so as to give four sides for dissipation of the heat, each block being separated from the adjacent block by a 2-ft open slot. To determine the value of cooling pipes, two of the blocks, namely, 27 and 31, on the south side of the river were built with vertical pipes extending their full height, through which water was pumped to assist in cooling the concrete. (See Fig. 6(*b*) for location of blocks.)

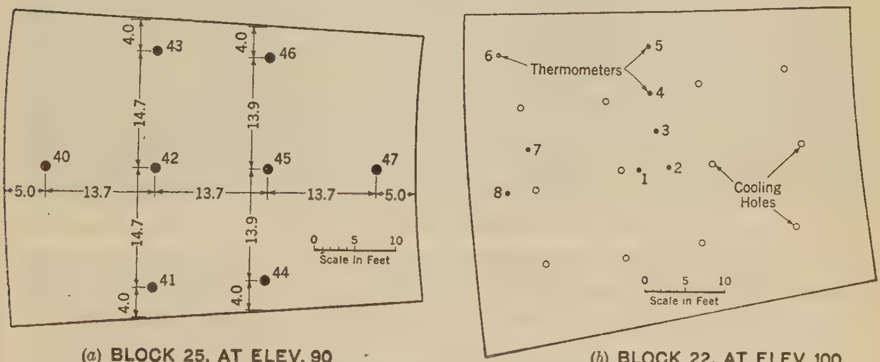
Thermometers, Provisions for Cooling, and Tests of Effectiveness

At Ariel a rather complete test program was undertaken to determine: (*a*) The maximum temperature in the concrete; (*b*) the rate at which internal heat would be radiated from uncooled blocks; and, (*c*) the rate at which internal heat would be radiated or removed by cooling water from artificially cooled blocks.

As mentioned previously two of the arch blocks, namely, 27 and 31, were provided with vertical pipes for circulating cooling water. It was desirable to have a complete picture of the maximum temperatures in the concrete and the rate of temperature drop in order to check the assumptions made in the design, and, therefore, to furnish reliable information for future designs of a similar character. The effectiveness of the cooling facilities in these two blocks furnished information as to the provisions that had to be made in the last three blocks of the arch. It was necessary to keep these three blocks low until late in the spring of 1930 in order to help pass possible large floods. It was known that it would be necessary to build all three of them up rapidly and it was absolutely essential to reduce their internal temperature as much as possible in not more than two months' time.

Beginning with the placing of concrete in the arch in June, 1930, temperature readings were initiated in the thrust-block, and in a considerable number of the arch blocks. For this purpose resistance thermometers were embedded in the concrete and the leads were connected in groups to convenient terminal boards. Out of a total of 289 thermometers embedded all but 34 were in perfect working order at the time the arch was closed in April, 1931, and practically all of them were still in working order on March 1, 1932. Several of them were checked against mercury thermometers and the readings were found to agree within an average of 1° C. In computing the temperatures from the readings of the resistance thermometers allowance was made for the resistance of the leads as well as for the resistance of the connection to the test set.

Two methods of placing the resistance thermometers in a plane in a block were used. The first, shown in Fig. 13(*a*), was to place them in three parallel lines up stream and down stream across the block. The instruments on the middle one of these three lines were placed on either side of the center of the block. This method brought only two instruments near the center of the block, but provided a good arrangement for securing temperatures after the slots were filled. The second method (see Fig. 13(*b*)), was to place one



(*a*) BLOCK 25, AT ELEV. 90 (*b*) BLOCK 22, AT ELEV. 100

FIG. 13.—LOCATION OF RESISTANCE THERMOMETERS IN BLOCKS.

set of thermometers on a line extending up stream and down stream to the center of the block with another row at right angles to this, or lengthwise of the dam. In this case an additional thermometer was usually placed in a corner of the block about 5 ft from either face. This arrangement was not as satisfactory as the first one after the slots were filled and the water began to rise against the dam. However, the number of thermometers used was sufficient to overcome the deficiencies of either arrangement and to allow consistent records to be obtained. Table 5 shows in what blocks and at what elevations the resistance thermometers were placed. It also shows in what blocks cooling holes were built.

TABLE 5.—Location of Thermometers and Cooling Facilities

Block No. (See Fig. 6)	Number of thermometers	Elevation of thermometers	Cooling equipment	Remarks
17...	26	210–190–170... 150–130–105...	....	Part of thrust-block
20...	19	210–160–120... Slot 25–26–32..	Slots to Elevation 100; twelve 5-in. cored holes..........	Rapid construction; raised 130 ft in 23 days
21...	22	210–150–110...	Twenty 5-in. cored holes; ten carried to top	Rapid construction; raised 142 ft in 14 days
22...	22	210–150–100*..	Twelve 5-in. cored holes; and one slot	Rapid construction; raised 152 ft in 23 days
25...	44	90–60–30*..... 5 and –40......	....	Highest block in dam; Elevation–73.7 to Elevation 240
26...	36	220–190–160... 130–100.......	....	....
27...	41	220–190–160... 130–100.......	Six 10-in. steel pipes.......	Very effective
31...	21	220–190–160...	One 12-in. steel pipe........	The first test block
32...	37	220–190–160... 130–100.......	....	A typical uncooled block
......	268	....	....	Total

* See Fig. 13.

It has been pointed out already that the fast construction schedule adopted for building the Ariel Dam required unusual steps to be taken in order that the arch might be closed without having temperatures appreciably higher than those adopted for the design. The greater part of the required cooling was accomplished by limiting the cement to 1 bbl per yd and by building the arch in blocks 30 ft long, with 2-ft slots between. The influence of the slots was variable, depending upon the elevation. Toward the top of the arch where the thickness was only about 20 ft the additional area at the ends of the blocks was smaller than the areas of the up-stream and down-stream sides of the arch block. At lower elevations, where the arch attained a thickness of 60 ft, or more, the additional area was much more than that of the two up-stream and down-stream faces of the 30-ft blocks. In the upper thin part of the arch the additional area was not so much needed. At the lower thick part of the arch, where it was needed most, the additional area gained was greatest.

Blocks 20, 21, and 22 (Fig. 6 (*b*)), which were left low through the fall and winter of 1930-31 to act as an emergency flood spillway, were raised one at a time as rapidly as possible. For example, Block 20 was raised 130 ft in 23 days; Block 21 was raised 142 ft in 14 days; and, Block 22 was raised 152 ft in 23 days.

It was known in advance that this would be the construction procedure, and experiments and observations in other blocks made it possible to determine the cooling facilities required in these three critical blocks. Vertical 5-in. cored holes were adopted. These holes were made by means of a 5-in., hollow, rubber conduit form which was raised at intervals as the concrete came up.

Block 31, well up on the left side of the arch, was built with one 12-in. cooling pipe up the center, and water was circulated through it continuously.

Block 27, near the left bank of the river, was equipped with six 10-in. pipes. Two of these pipes extended vertically from Elevation 90 to Elevation 170; two to Elevation 180, and the remaining two to the top of the block. These pipes were located in three rows of two each. The two pipes were 10 ft apart and the rows of pipes about 9 ft apart. The temperature records from Blocks 27 and 31 through which water was circulated continuously, gave information as to what could be accomplished by this method of cooling, using openings of different sizes and number. Fig. 14 shows one of the rubber conduits being raised.

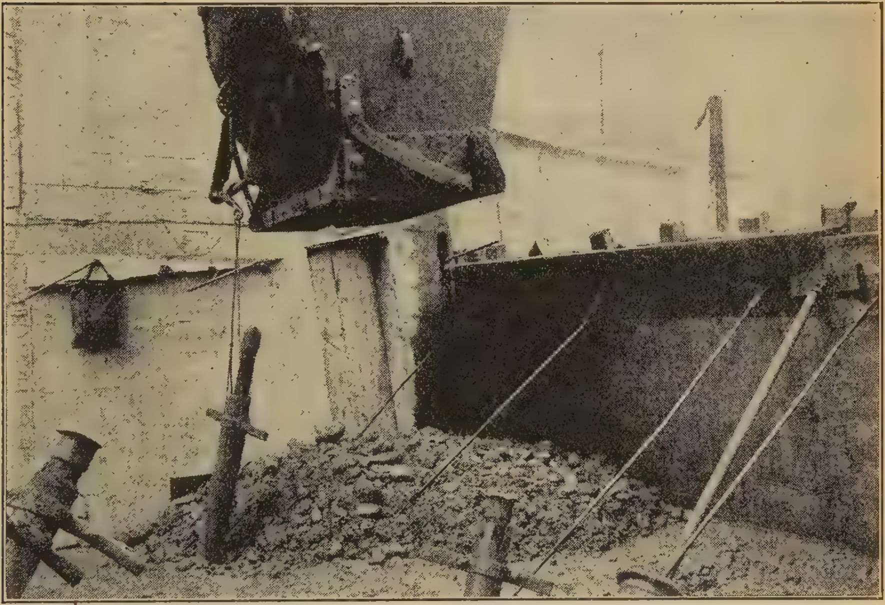


FIG. 14.—FIVE-INCH HOLLOW RUBBER CONDUIT BEING RAISED.

The circulating water was admitted to the bottom of the pipes, completely filled the vertical openings through the blocks, and escaped through overflows so that the heat was entirely removed by thermal conductivity without any reliance on evaporation. The possibility of cooling by the latter process was considered and might have been utilized by releasing a small quantity of water into the holes and inducing a natural or forced draft. It was thought at the time that this method would not be as effective as the one adopted and would be more difficult to regulate properly.

To determine the rate of heat transfer from the concrete to the water in the cooling openings the circulation was shut off in one of the blocks and the temperature at different depths in the hole was read by a series of resistance thermometers over an interval of time. The readings were continued until there was a notable decrease in the rate of temperature rise in the water.

Fig. 15 shows the arrangement of the 12-in. cooling pipe in Block 31, and Fig. 16, the temperature measurements made on three different dates. With water flowing in the cooling pipe at its normal rate, temperatures of the water were measured at the locations of the four resistance thermometers (Fig. 15). Then, with the water turned off (no overflow) time-temperature readings were taken. A tangent to the time-temperature curve at the time water stopped flowing through the cooling pipe gives a means of determining the rate of heat flow from the concrete to the cooling pipe; that is, tangents to the three curves (Fig. 16) at the point of origin show the rate at which the temperature of the water rises. It will be noticed that there was a marked decrease in the rate for readings taken subsequent to October 8. This was due, no doubt, to the fact that the block had cooled off considerably by the time the last readings were taken on December 6. Tests similar to this one were made in the other cooling holes and, from the data thus obtained, a diagram was made to show the amount of heat removed, in kilogram-calories per hour per foot of hole. These results are shown by Fig. 17.

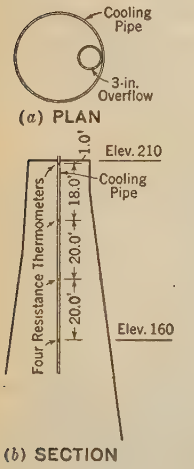


Fig. 15.—Arrangement of 12-Inch Cooling Pipe in Block 31.

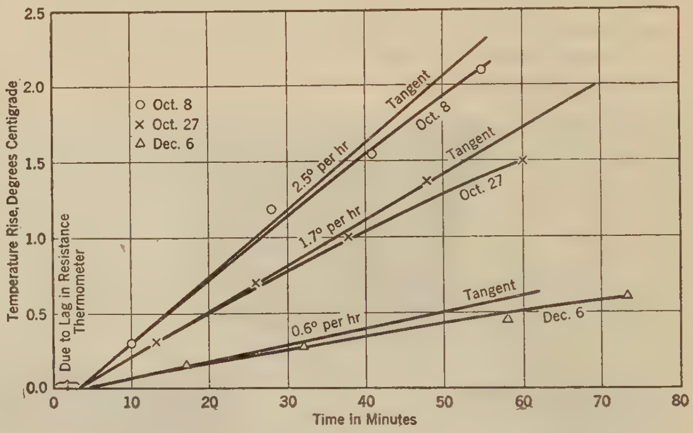


Fig. 16.—Rise of Temperature of Water in Holes.

The readings obtained were not strictly comparable as the cooling holes passed through concrete of different ages so that the temperature gradients to the holes were not the same. Allowance has been made for these variables, however, so that the curves probably represent the rate of cooling that could be expected from the number and sizes of openings shown when used in a manner similar to that at Ariel Dam. It will be noted that the larger pipes removed considerably more heat than the smaller ones for the same difference in temperature between the cooling water and the maximum temperature of the block. As a result of these experiments, Block 20 was built with twelve 5-in. cored holes through it, Block 21 with twenty 5-in. holes (of which ten only extended to the top), and Block 22 with twelve 5-in. holes. It will be recalled that these were the blocks to be built last and that they were raised 130 to 152 ft in 14 to 23 days. The actual reduction in the temperature of the concrete, accomplished by circulating water through the openings in the various blocks, is shown by Figs. 18 and 19.

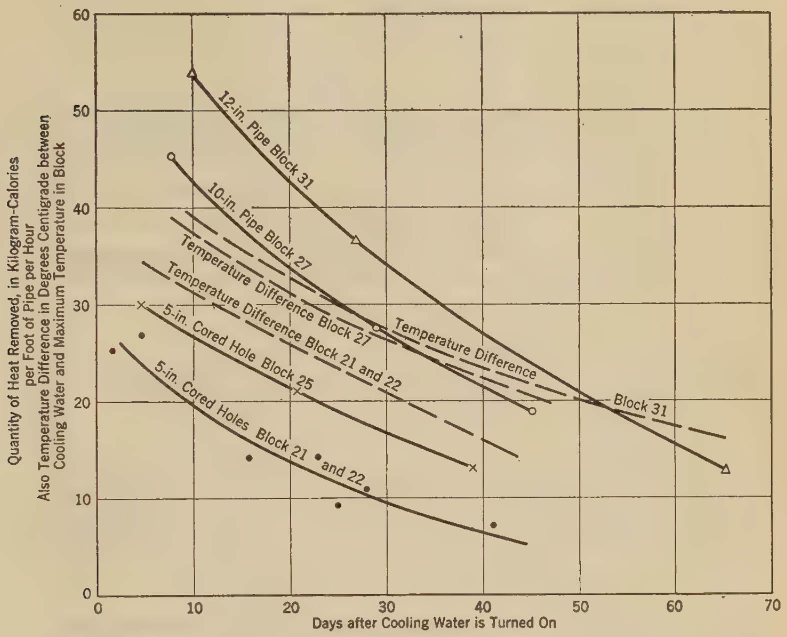


FIG. 17.—RATE OF HEAT REMOVAL BY COOLING WATER.

Referring to Fig. 19(*a*): Block 21 contained twenty 5-in. cored cooling holes and was built without slots; Block 27 contained six 10-in. cooling pipes; Block 25 had a slot (on one side only), at Elevation + 5; at Elevation − 40, Block 25 had no slots, but is confined between rock walls; Block 32, at Elevation 100, is affected by its proximity to the foundation rock; and all other blocks are normal.

Referring to Fig. 19(*b*): Block 31 contained one 12-in. cooling pipe at the center; Block 27 contained six 10-in. cooling pipes; and Block 21 contained twenty 5-in. cored holes, and was placed between Blocks 20 and 22 without slots.

Referring to Fig. 19(*c*): Block 27 contained two 10-in. cooling pipes, and Block 31 contained one 12-in. cooling pipe.

In addition to the artificial cooling of a few of the blocks by circulating water through them, all surfaces of the arch, as well as those of the remainder of the dam, were sprinkled continuously with water. Evaporation of this water assisted somewhat in the removal of heat from the concrete and, to a large extent, prevented surface checking and cracking.

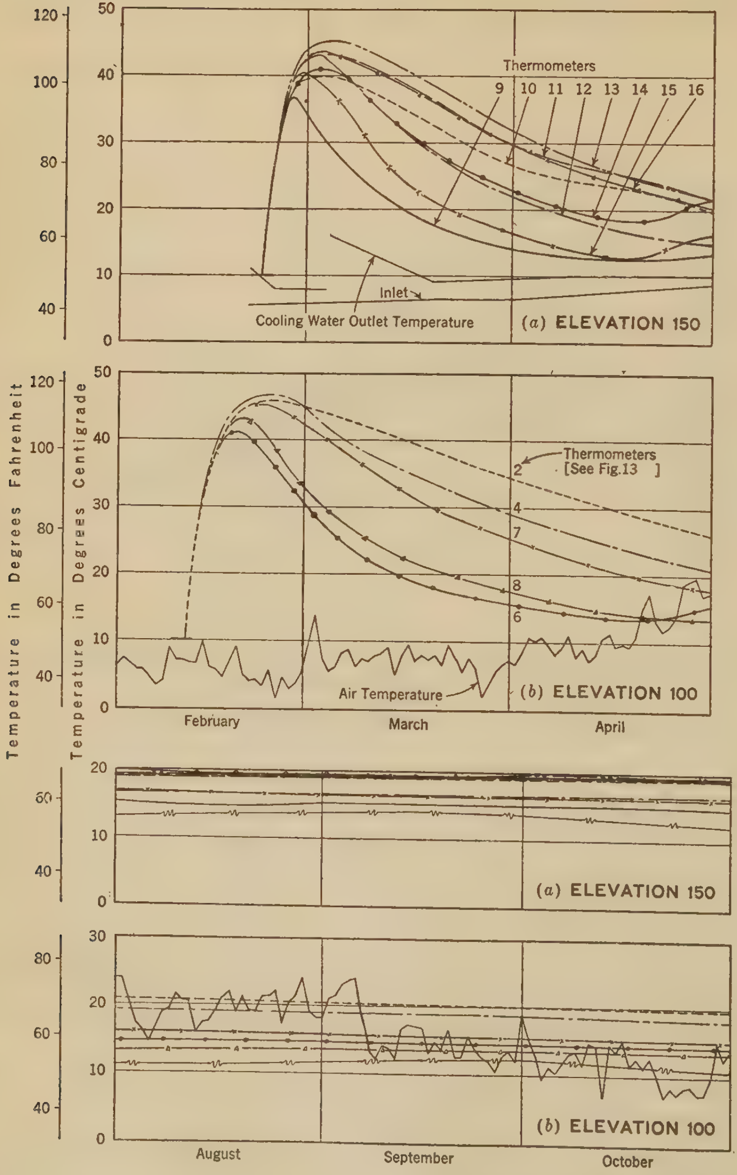
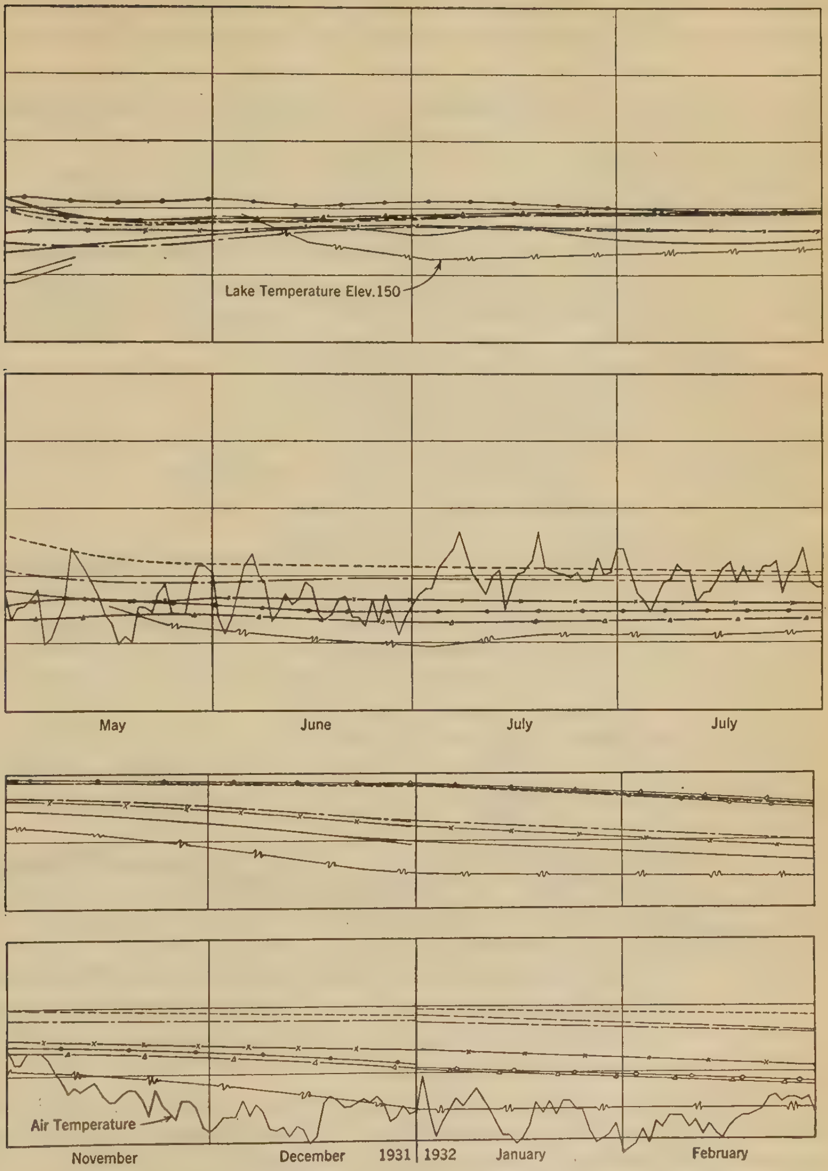


FIG. 18.—TEMPERATURE RECORDS



for Block 22 (Artificially Cooled).

TEMPERATURES IN TYPICAL ARCH BLOCKS

The temperatures developed in the thrust-block and also in the arch blocks are recorded on seventy drawings which were kept up to date from the time the earliest thermometers were placed and readings began in May, 1930.

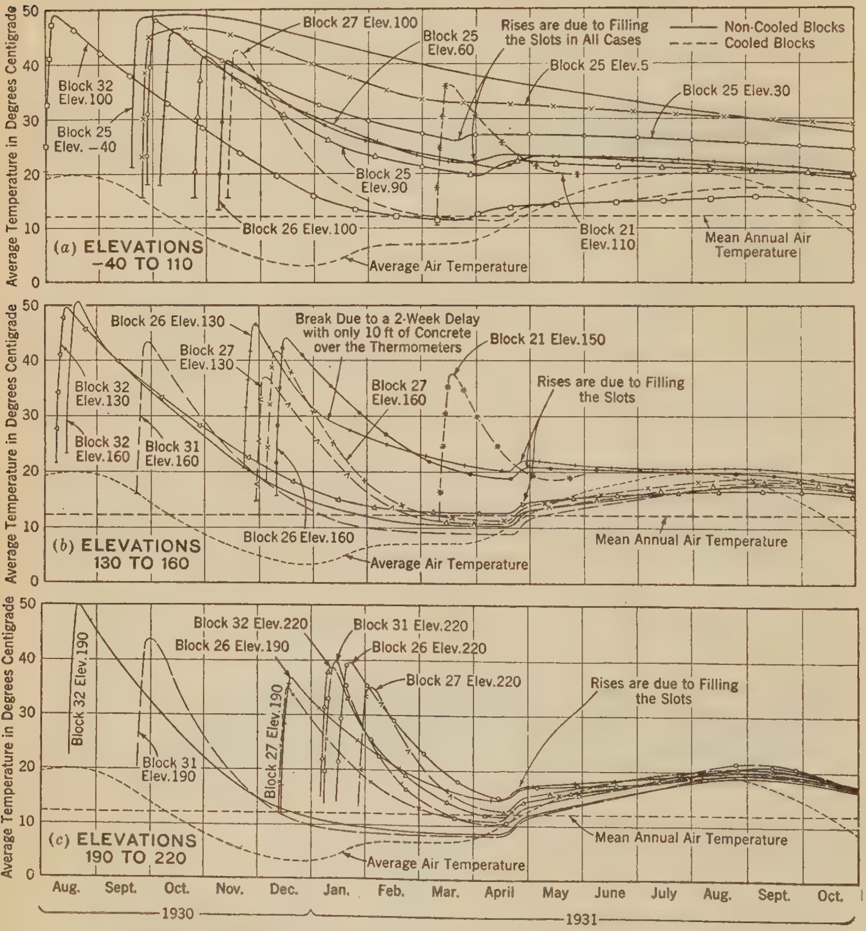


FIG. 19.—AVERAGE TEMPERATURES IN VARIOUS BLOCKS.

It is not feasible to present all these records and, fortunately, this is not necessary as the results are very similar for the different blocks so that typical examples will convey the necessary information.

The 289 electrical resistance thermometers previously mentioned were placed in the thrust-block and in nine of the arch blocks. This put an average of twenty-nine thermometers into a block, located in three to six horizontal planes.

The first thermometers were placed in the lower part of the thrust-block and readings were begun about May 1, 1930. The next thermometers were placed at five different elevations in Block 32 located near the south end of the dam and readings were begun about August 1, 1930. Another set of thermometers was placed in Block 31 which was built with the one 12-in. cooling hole. The readings from thermometers in this block began on September 30, 1930. Thermometers were placed in Block 27 in which the six 10-in. cooling pipes were placed, and readings were begun on November 12, 1930. Thermometers were placed in the lower part of Block 25 and readings were begun on September 15, 1930. In Block 26, thermometers were placed at five different elevations, and readings were begun on November 20, 1930. In each of Blocks 20, 21, and 22, which were built with the 5-in. cored holes for artificial cooling, three sets of thermometers were placed at different elevations, and readings were begun in the early part of February and March, 1931.

Fig. 18 shows the temperatures at Elevations 100 and 150 in artificially cooled Block 22 from the time the readings began on February 12, 1931, to March 1, 1932. As noted, this block was built rapidly and had twelve 5-in. cored holes in it. The results shown for this block are typical of the results in Blocks 20 and 21 which were also built up rapidly at approximately the same time, and were also artificially cooled.

Records of the concrete temperatures in the uncooled Block 25 are given by Fig. 20, which shows the temperatures at Elevation 90, from October 1, 1930, to November 1, 1932. This example is typical of the results obtained in the uncooled blocks. No readings were possible on Thermometers 40, 41, and 43, probably because of broken leads. The positions of the thermometers are shown in Fig. 13(*a*).

The temperature data obtained from the thermometer readings in all the blocks have been used to compute the "average" temperature of each block at each elevation where thermometers were placed. These "average" temperatures, extending from the time readings were begun in each case until November 1, 1931, are shown by Fig. 19. The curves for the uncooled blocks are shown by full lines, while those for the cooled blocks are shown by dashed lines. These curves show clearly the story of the rapid gain in heat, the maximum temperatures reached, and the slow but varying loss of this heat with the lapse of time, depending upon whether the block was cooled or uncooled. There is also indicated the average air temperature during this period and the date on which the slots were filled. These three sets of curves, as well as Figs. 18 and 20, show plainly the effect of filling the slots in April, 1931.

The following statements are based on a study of these temperature records, some of which extend over a period of twenty-two months:

1.—Rapid placing of the concrete, particularly in thick layers, caused high temperatures.

2.—The maximum temperature was 61.9° C (143.4° F).

3.—The highest placing temperature was 23.3° C (74° F).

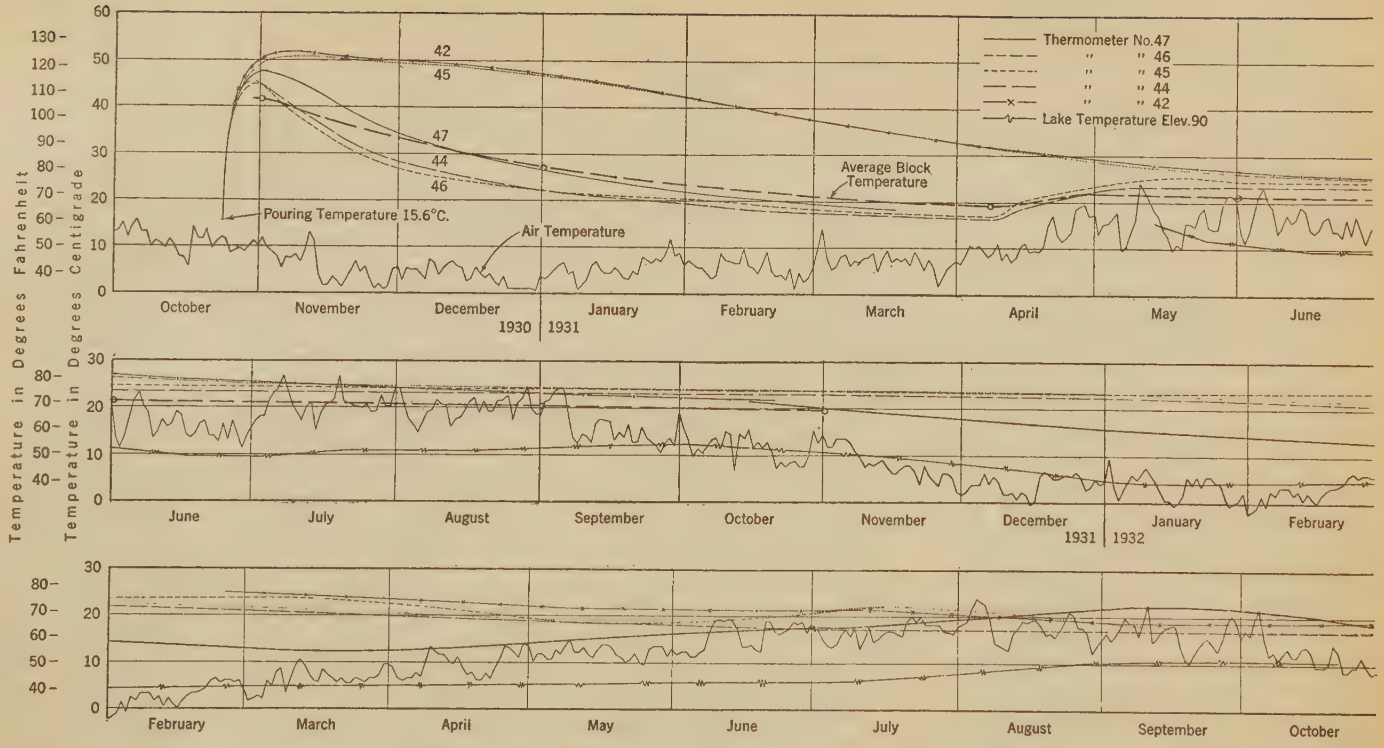


FIG. 20.—TEMPERATURE RECORDS OF BLOCK 25 (NOT COOLED) AT ELEVATION 90.

4.—The highest "average" temperature of a lift was 51.0° C (123.8° F) for an uncooled block.

5.—The usual temperature without cooling was about 50° C (122.0° F).

6.—The lowest "average" temperature of a lift was 35.0° C (105.0° F) for the block cooled with six 10-in. pipes.

7.—The average rise in temperature was 23° to 28° C (41.4° to 50.4° F).

8.—Due to favorable radiation conditions the concrete within 4 ft of exposed surfaces never attained a high temperature.

9.—Temperature strains due to the unequal cooling of the exterior and interior of the concrete are severe enough to cause surface cracking of a large block if it is not reinforced, or unless the interior of the block is cooled artificially.

10.—Cored holes or a system of pipes through which cooling water can be circulated are practicable from a construction standpoint and are very effective in reducing the heat of the concrete if speed of construction is desirable.

11.—In spite of the high-speed construction schedule the temperature of the various blocks was satisfactory at the time of closing the arch in April, 1931 (see Table 6). For several different elevations the "average" temperature conditions (see Table 7) were, as follows:

TABLE 6.—ACTUAL TEMPERATURES IN ARIEL DAM AT TIME OF CLOSING, IN DEGREES CENTIGRADE

Block No.	18	19	20	21	22	23	24	25	26	27	28
Elevation:											
220	10.7	11.2	18.0	20.0	17.0	13.5	15.0	14.9	15.0	12.7	14.8
190	10.7	11.2	18.0	21.0	17.3	14.8	14.4	14.4	14.2	11.0	13.4
160	13.0	14.0	18.0	23.0	18.0	18.2	18.4	18.9	18.7	11.0	17.3
130	12.7	12.6	21.0	25.0	18.5	20.8	20.4	20.3	20.1	10.4	19.5
100	12.7	12.7	16.0	19.0	19.0	20.0	20.0	20.6	20.5	11.2	20.0
90	....	....	....	18.0	19.0	19.6	19.0	19.6	19.3	11.2	19.0
60	....	....	....	11.5	11.0	21.6	13.0	21.6	13.0	23.0	12.0
30	....	....	....	....	....	26.0	26.0	26.0	26.0	....	....
5	....	....	....	....	....	....	32.0	33.0	31.0	....	....
−40	....	....	....	....	....	....	....	38.5	....	....	....

Block No.	29	30	31	32	33	34	35	36	37	38
Elevation:										
220	11.9	11.8	10.7	11.7	11.6	8.7	8.7	8.5	8.7	8.7
190	9.8	8.7	8.0	8.6	8.6	8.7	8.5	8.5	8.6	....
160	12.7	10.4	8.6	10.2	10.2	10.0	10.0	10.0	....	....
130	14.8	12.7	12.7	12.7	12.7	11.0	11.0	11.0	....	....
100	14.5	12.4	12.7	11.4	11.4	11.4	11.4	....	....	....
90	14.8	12.7	12.7	....	....	....	....	....	....	....
60	14.8	....	....	....	....	....	....	....	....	....
30	....	....	....	....	....	....	....	....	....	....
5	....	....	....	....	....	....	....	....	....	....
−40	....	....	....	....	....	....	....	....	....	....

From Elevation 240 to Elevation 190 the temperature of the concrete ring was down to about the average annual temperature (12° C).

From Elevation 190 to Elevation 60 the temperature was not more than 5° C above the average annual temperature.

From Elevation 60 to Elevation − 73 the temperature was considerably higher than the design value, but in this zone the arch is short and functions more nearly like a wedge.

TABLE 7.—TEMPERATURE DROPS AFTER CLOSING THE ARCH, IN DEGREES CENTIGRADE

Elevation	Average temperature of arch at closing, April 18, 1931	Drop in temperature to March 1, 1932	Drop in temperature below annual mean for which dam was designed
220	12.7	9.3	6.1
190	12.1	7.4	5.2
160	14.5	5.7	4.2
130	16.0	4.0	3.3
100	15.7	2.2	2.2
90	17.0	1.8	2.0
60	15.7	0.3	1.7
30	26.0	...	1.6
5	32.0	...	1.4
–40	38.5	...	1.3

The relative economy of building the dam with construction joints only and with embedded cooling pipes, as compared to building it as a series of blocks with slots between, is somewhat debatable. At Ariel, the Construction Organization was inclined to believe that the first plan would have been cheaper.

Due to the arrangement of slot construction the longitudinal shrinkage of the arch after closure was extremely small.

In connection with the temperature records it should be noted that the outside surfaces exposed to the sun were kept wet constantly by sprinkling. This sprinkling brought the temperature down about 5° C, or to about air temperature. Due to evaporation those parts of the surface in the shade had a temperature slightly lower than air temperature. The records indicate that the residual setting heat will not be entirely dissipated for two or three years.

It may be noted that the concrete temperatures began to go down again about October 1, 1931, following the high rise during the summer months which commenced with the filling of the slots in April. These temperatures follow nearly parallel to average daily air and reservoir water temperatures.

DEFLECTIONS OF THE ARCH DAM

Actual measurements of the deflections of full-sized arch dams are few. Observations of this type have been made on the Stevenson Creek Test Dam.[2a] Concrete models of the Stevenson Creek and Gibson Dams have been tested, the latter in connection with the design of the Boulder Dam. A plaster-celite model of the Boulder Dam itself was tested in 1932.[3]

For the purpose of determining the deflection of the Ariel Arch twenty-one points were set in the arch and gravity abutment sections. Measurements were made: (1) After the slots were filled and just before water loading began; (2) at intervals during the time the reservoir was filling; and (3) at monthly intervals during 1931 and bi-monthly intervals during 1932, to determine the maximum deflection and the seasonal variation therein.

[2a] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.

[3] "Model Tests Confirm Design of Hoover Dam," by J. L. Savage and Ivan E. Houk, Members, Am. Soc. C. E., *Engineering News-Record*, April 7, 1932.

The Ariel Arch was not subjected to its water load until it was completed. This is a highly important consideration if stresses are to be expected, in different parts of the arch, in accordance with the calculated values.

The targets were brass plugs set into the concrete. The pointed, projecting part was painted with white enamel, and $\frac{3}{8}$-in. blackened holes on each vertical face were used as sights. Fig. 6(*b*) and Fig. 21 show the location of the various deflection points on the arch and thrust-block. The deflections were read by means of horizontal angles between fixed targets and the arch targets, from an instrument base on either side of the river. These two bases were tied together by a triangulation system carefully laid out between them and a base line on the right bank of the river.

Nearly all the transit bases and fixed targets were on rock. The readings were made with a 20″ transit with a special base. The transit plates and the

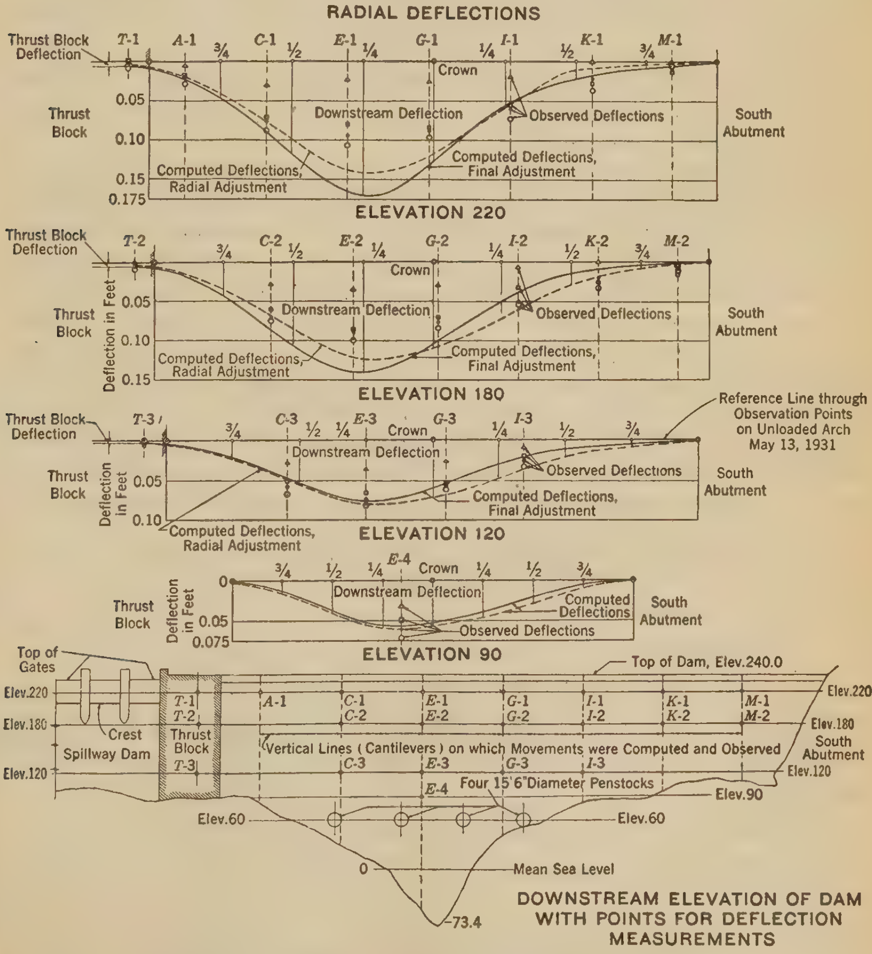


FIG. 21.—COMPUTED AND MEASURED RADIAL DEFLECTIONS.

fixed base plates were match-marked in order to obtain the same set-up each time. The targets were arranged so that when turning away from any target on the dam to the fixed point no focusing was necessary and the telescope was not turned about the horizontal axis. The angles were read by repetition, 10 to 20 hr being required to take a complete set of readings on the twenty-one points. The measurements taken on May 11 and 12, 1931, just before the reservoir began to fill, were considered the "zero" readings. Fig. 21 shows the radial, and Fig. 22, the tangential, deflections for various dates to July 15, 1932.

The computed deflections are for water at Elevation 240; they have been increased to include the deflection of the thrust-block. The dam was closed in May, 1931, and the readings taken at that time are considered as the zero readings or reference points for all future readings. Deflections were read monthly during 1931 and every two months during 1932. Only four sets of these readings are shown in Figs. 21 and 22. Maximum deflections occurred in May, 1932, and subsequent deflections decreased as the temperature of the structure increased. In Fig. 22 clockwise displacements indicate movement toward the south abutment and counter-clockwise displacements indicate movement toward the thrust-block.

Table 8 shows a comparison of the actual deflections from July 12 to 15, 1932, and the calculated deflections. It will be seen from this table and from Figs. 21 and 22 that the measured deflections are considerably less than calculated at the center of the arch, but slightly more toward the abutments. The calculated deflections are for water at Elevation 240, whereas for the measured deflections the water was at lower elevations. This partly explains why they have not as yet reached their maximum. Another reason is that the concrete had not yet reached the design temperature, except above Elevation 190 (see Table 6). A third reason is the fact that the modulus of elasticity of the foundation and abutment rock probably is not exactly the same as that assumed in the design. The maximum yearly deflection is expected to occur about the last of April of each year, as this is the time when

TABLE 8.—COMPARISON OF CALCULATED AND MEASURED RADIAL ARCH DEFLECTIONS

Deflection points (See Fig. 6 (*b*))	*T*	*A*	*C*	*E*	*G*	*I*	*K*	*M*
Elevation 220:								
Computed	0.0073	0.023	0.089	0.164	0.132	0.053	0.018	0.006
Measured	0.0110	0.029	0.088	0.108	0.098	0.073	0.038	0.008
Elevation 180:								
Computed	0.0055	0.020	0.087	0.140	0.098	0.039	0.011	0.003
Measured	0.0100	.....	0.075	0.100	0.084	0.053	0.033	0.011
Elevation 120:								
Computed	0.0038	0.013	0.047	0.077	0.054	0.022	0.008	.....
Measured	0.0010	.....	0.069	0.082	0.062	0.033	.....	.....
Elevation 90:								
Computed	......	.....	.....	0.058	.....	.....	.....	.....
Measured	......	.....	.....	0.070	.....	.....	.....	.....
Elevation 60:								
Computed	......	.....	0.020	0.040	0.023	.....	.....	.....
Measured	......	.....	.....	.....	.....	.....	.....	.....

the arch is expected to reach its lowest average temperature. It is quite probable that the maximum deflection will not be reached before April, 1933.

Recorded observations on the behavior of the arch for fourteen months (to July 15, 1932), indicate that:

1.—The maximum recorded deflection (in May, 1932) is 0.108 ft (1.3 in.) at Elevation 220 in Block 25.

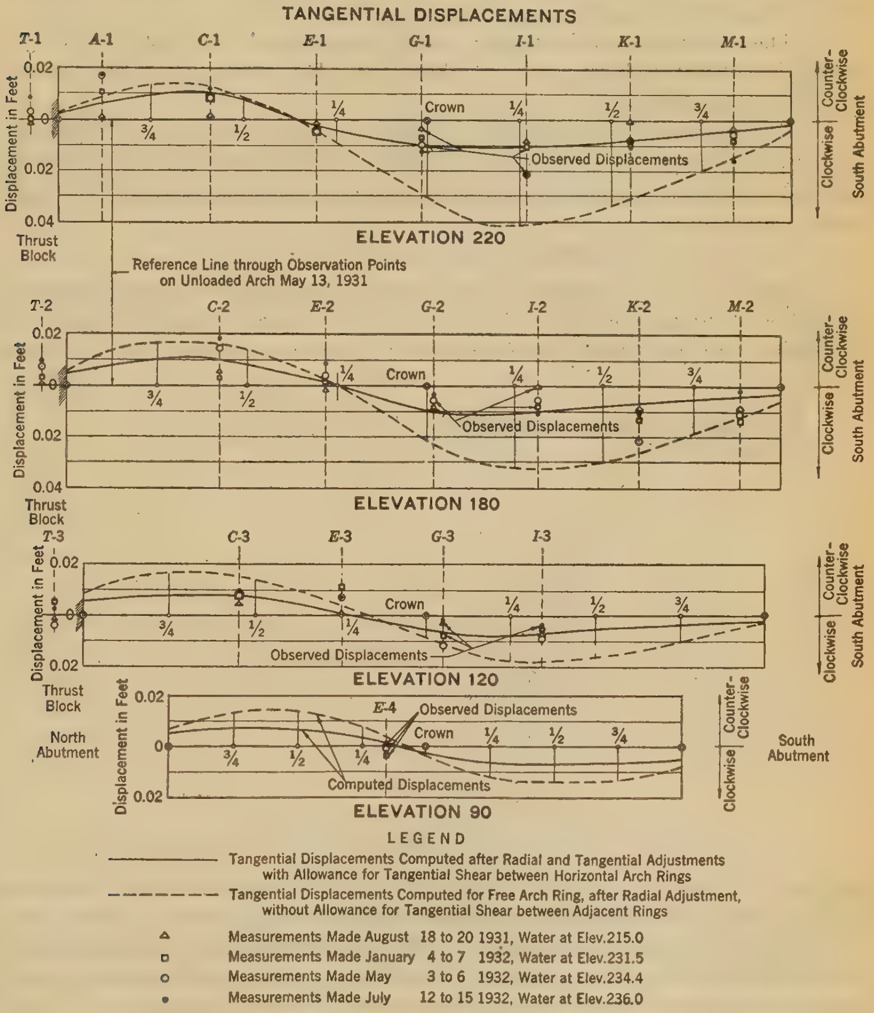


FIG. 22.—COMPUTED AND MEASURED TANGENTIAL DEFLECTIONS.

2.—The deflections at Elevation 180 are slightly less than those at Elevation 220.

3.—From the maximum point the deflections taper off smoothly to the abutments and foundations.

4.—Closing the slots did not cause any measurable up-stream deflection of the arch. The expansion due to heating of the concrete in the slots apparently was taken up in closing radial cracks, compression of the concrete, and by restraint of the foundations.

5.—The concrete thrust-block, which constitutes the north abutment of the arch, shows a slight tangential movement and there has also been a small down-stream radial movement of the top, amounting to 0.011 ft (in May, 1932), compared to the calculated value of 0.0073 ft.

6.—The measurements indicate a radial movement of the south abutment (which is natural rock in place) of about twice that assumed in the design, or a little more than 0.01 ft.

7.—The measured tangential displacements agree closely with those calculated and indicate, by direction and amount, that arch action took place as expected, with the maximum movement near the quarter-points of the arch rings.

VERTICAL SHRINKAGE OF CONCRETE

In order to obtain additional data on the shrinkage of the arch concrete, the vertical shrinkage of uncooled Block 25 was measured from the time of its completion late in January, 1931, to late in November, 1931. Fig. 23 shows the height of this block. The elevation of Point *E-O* was computed from the relation between the horizontal distances shown and the distance between the non-level line of sight and Point *E-O*. The accuracy of the measurements is probably of the order 0.005 to 0.01 ft. For the first seven months the measured shrinkage was very close to the computed value (based on a coefficient of contraction of 0.0000056 per degree Fahrenheit and temperatures taken from thermometers in the upper parts of Blocks 25 and 26). For the last three months the difference increased gradually, the measured shrinkage at the end of November being 0.046 ft and the computed, 0.031 ft. Of this shrinkage, 0.024 ft took place before the slots were filled, at which time the average

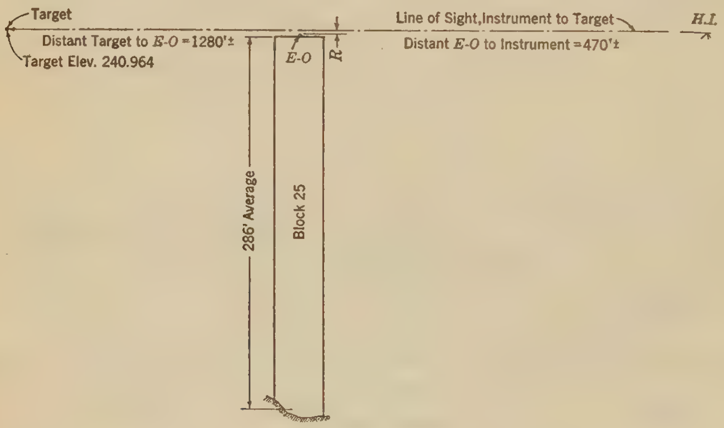

Target
Distant Target to E-O = 1280'±
Target Elev. 240.964
Line of Sight, Instrument to Target
H.I.
Distant E-O to Instrument = 470'±
E-O
R
286' Average
Block 25

FIG. 23.

temperature of the block was 20.2° C, as compared to 12° C, the average annual temperature expected ultimately. Further cooling has not resulted in diagonal or other cracks.

General Observations

In addition to the observations which have been listed in considerable detail, other observations were made which will be of interest to engineers called upon to design concrete structures of considerable magnitude. These observations were made with care, but space does not permit indicating in detail the procedure that was followed:

1.—A test was made to determine whether the holes cored in the concrete for cooling purposes would cause radial cracking. An experimental block was tested and careful examination did not disclose any cracks and a light wire spiral showed no indication of change in electrical resistance which would have been the case if cracks of any size had developed.

2.—The specific heat of saturated concrete was found to be 0.26, while the specific heat of oven-dried concrete was 0.21. It is believed that the higher figure should be used in determining the heat released in a dam.

3.—Tests were made to determine the amount of heat liberated by the concrete after it was in the forms. It was determined that under average conditions 114 calories per gram of cement had been released at the end of 45 days.

4.—As a result of pouring some concrete in this arch dam with cooling facilities and some without cooling facilities, it was found that the placing of fresh concrete at a low temperature had two distinct advantages: First, the temperature rise due to setting was so much slower that radiation and artificial cooling had more time to take effect; and, second, since the placing temperature was lower the maximum temperature was correspondingly lower.

5.—Facilities were provided for measuring hydraulic uplift on the base and at three different horizontal days' work joints of the thrust-block. The maximum observed uplift between the rock and the base of the dam was less than 20% of the static head and this was within 5 ft of the up-stream face. No uplift occurred on the days' work joints, indicating that water-tight joints can be secured by proper design and careful construction.

Conclusions

It is believed that the many facts determined from the extensive tests at the Ariel Dam warrant the following conclusions:

1.—By adopting proper construction methods an arch dam may be built at high speed and the temperature of the concrete reduced to a suitable figure at the time of arch closure. This end may be attained by the following means:

(*a*) By limiting the quantity of cement used. An excess of cement over that required for proper strength and quick setting works against the attainment of low concrete temperatures. The proportioning of the aggregate and the water-cement ratio must also be watched constantly.

(*b*) Construction in comparatively short blocks will more than double the radiating surface in the thicker parts of the arch and will greatly increase the rate of temperature drop in the interior of the concrete.

(*c*) The use of cooling pipes or holes cored in the concrete is entirely practicable from a construction standpoint, and the circulation of cold water through these openings will assist in materially reducing the concrete temperature by increasing the rate of heat removal. This artificial cooling will also aid in the prevention of surface cracks. Where artificial cooling is used, it is essential that it begin as soon as possible after the concrete is placed.

2.—Hollow rubber cores make excellent forms for cooling holes and they eliminate entirely the possibility of grout and débris collecting in the holes. Steel pipes are entirely suitable for forming the holes, but are not as convenient as the cores.

3.—Electrical resistance thermometers of a high degree of reliability can be obtained. These thermometers can be placed in the concrete and the leads can be brought out without interfering materially with the concreting work or adding to its cost.

4.—If necessary the concrete can be placed rapidly in large quantities and to a great depth without sacrificing any of its desirable qualities. In the Ariel Dam one block was built up 125 ft in 10 days. High temperatures in the interior of a block, 40 by 90 ft in plan, were not detrimental to the strength of the concrete.

5.—The temperature of the concrete in parts of the structure eighteen months after the construction began is still (1933) somewhat above the annual average air temperature, but is slowly approaching that figure. The maximum observed temperature in the concrete was 61.9° C (143.4° F) and the average temperature rise varied from 23° to 28° C (41.4° to 50.4° F).

6.—Diamond drill cores taken from the dam at several places indicated that a saturated condition of the concrete existed fifteen months after placing. These cores also indicated an increase of strength from the surface toward the interior, the concrete in the interior having a compressive strength as great as, or greater than, that indicated by the regular test cylinders of equal age.

7.—A measurable vertical shrinkage took place in the concrete and if it had not been for the slot construction there would have been a high concentration of loads at the quarter-points of the dam where the steep sides of the canyon occur. The vertical shrinkage subsequent to the closing of the arch has not caused the diagonal cracks which have occurred in some dams.

8.—There is an entirely satisfactory agreement between the actual and the calculated radial and tangential deflections, both in direction and in amount. Measurements show a down-stream movement of the top of the thrust-block of 0.0037 ft more than calculated.

9.—Annual temperature changes account for more than 30% of the deflection of the Ariel Arch Dam. If a drop in seasonal temperature has such an effect on the dam it is evident that the chemical heat developed in the concrete should be removed, as far as possible, in order to avoid additional deflection and stress.

10.—The leakage under the dam is extremely slight. This can be accounted for by the particular care taken in the preparation of the foundations and the careful grouting of them for water-tightness.

ACKNOWLEDGMENTS

The Ariel Development was designed by the Engineering Department of Electric Bond and Share Company and was built by Phœnix Utility Company. The late Mr. H. F. Lincoln was Local Construction Manager, and was assisted by R. C. Booth, M. Am. Soc. C. E., as Assistant Construction Manager. The various temperature and deflection measurements were carefully carried out by R. W. Spencer, Assoc. M. Am. Soc. C. E., Resident Engineer, and I. L. Tyler, Assoc. M. Am. Soc. C. E., Assistant Engineer. Previous to the starting of design and construction, the project had been subject to study for some years under the direction of Lyman Griswold, M. Am. Soc. C. E., Consulting Engineer for the Northwestern Electric Company. D. C. Henny, M. Am. Soc. C. E., Consulting Engineer, at Portland, Ore., represented the State of Washington on design and construction matters in connection with the entire dam, and his wide experience was of value in considering the many special problems involved. L. T. Merwin, Vice-President and General Manager of the Northwestern Electric Company, took an active interest in making it possible to secure the information presented in this paper and in making it available to the Engineering Profession.

DISCUSSION

George Jacob Davis, Jr.,[4] M. Am. Soc. C. E. (by letter).—The author's Fig. 21 and the accompanying data seem to indicate that at Elevation 90 and Elevation 120 the actual deflections of the Ariel Dam agreed with the computed deflections reasonably well when the water was near its maximum elevation and the dam and the lake water temperatures were near their minimum values. At Elevation 180 and Elevation 220 the actual deflections, under the same conditions, were, roughly, three-fourths of the computed values at the crown and somewhat more than the computed values at the haunches of the arches.

At the lower elevations only a minor part of the water load is carried by arch action and the deflections, therefore, are governed largely by resistance to shear on a horizontal plane. At the higher elevations the deflection is due largely to bending.

It appears to the writer that the discrepancy between the actual and the computed deflections at the higher elevations may be accounted for by assuming that the modulus of elasticity of the concrete was actually much higher than the value of E_c = 2 000 000 lb per sq in., which was used in the design.

Figs. 8 and 11 show that at the time the maximum deflections were measured (May 3 to 6, 1932), the compressive strength of the concrete approximated 4 000 lb per sq in., and Fig. 10 indicates that in the interior of the mass of concrete the compressive strength may have averaged 5 000 lb per sq in., or more.

Stanton Walker, Assoc. M. Am. Soc. C. E., has shown[5] that the relation between the tangent modulus of elasticity, at 25% of the compressive strength, and the compressive strength of concrete may be expressed by the equation,

$$E_{t25} = 66\ 000\ S^{\frac{1}{2}} \qquad \text{(1)}$$

from which it may be surmised that if the strength of the concrete in the Ariel Dam was 4 000 to 5 000 lb per sq in., the modulus of elasticity was probably as high as 4 000 000 to 5 000 000 lb per sq in.

It is probable that the Ariel Dam, and all others thus far designed by the trial-load method, are considerably stiffer and stronger structures than they were assumed to be by their designers. Stiffer cantilevers would crack to a less depth, thereby changing the moment of inertia of the cantilever sections and decreasing the hydraulic uplift in the cracks, and stiffer arches would not crack so deeply at the crown and near the abutments. The distribution of the water load between the cantilever sections and the arch sections is, therefore, doubtless somewhat different from that assumed in the design.

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[4] Dean, and Prof of Civ. Eng., Univ. of Alabama, Tuscaloosa, Ala.

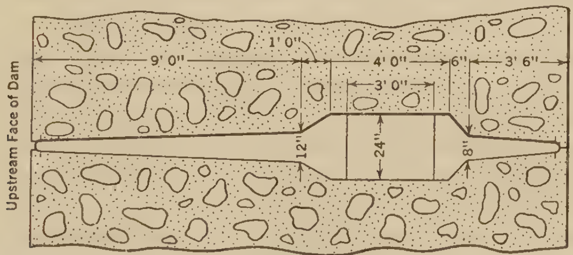
[5] "Modulus of Elasticity of Concrete", *Bulletin* 5, Structural Materials Research Laboratory, Lewis, Inst., Chicago, Ill.

Knowledge of the true value of the modulus of elasticity of the masonry is secondary in importance only to knowledge of the true value of its strength, if a rational design is to be made of an arch dam by the trial-load method. The effect of the moduli of elasticity of the materials used in the Stevenson Creek Dam and its foundations was studied by the Committee of Engineering Foundation on Arch Dam Investigation.[6]

It would seem that a careful study should be made also of the effect of the concrete in the core of the dam having a higher modulus of elasticity than that near the faces, for since the interior concrete had a higher strength value it probably also had a higher modulus of elasticity than the surface concrete. Such a difference in the compressibility of the concrete would affect the depth of cracking at the crown and abutments and, consequently, the moment of inertia of the section and the position of the line of thrust.

There may be some advantage in making the blocks of concrete shorter even than the 30 ft adopted in the construction of the Ariel Dam and also in proportioning the slots between the blocks so as to secure the benefits advocated[7] by A. V. Karpov, M. Am. Soc. C. E. It is also possible so to design the slots and to fill them in such a way as to compensate to a considerable degree for the shrinkage of the concrete in the blocks.

An attempt to attain this end was made, in 1928, by the writer in designing an arch dam, for the water supply of Tuscaloosa, on Little Yellow Creek, in Alabama. The blocks of the dam were made 50 ft long each side of a 4-ft "closing plug" which was not placed until the completion of the dam. At the ends of the two 50-ft sections, openings of the shape shown in Fig. 24 were left in the dam. These openings were provided with metal water-stops at the up-stream and down-stream faces of the dam. The openings were made as small as possible, so as to permit of their being completely filled with concrete in one continuous operation requiring only a few hours. They were large enough to permit a workman and an inspector to see that the concrete was properly placed through a tremie and spaded to avoid honeycombing. By this means the entire depth of concrete was placed before that at the bottom of the slot, or "expander joint", had set.



EXPANDER JOINT

FIG. 24

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[6] *Proceedings*, Am. Soc. C. E., May, 1928, Part 3, p. 153.

[7] *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 1309.

The hydrostatic head thus produced in the fluid concrete exerted sufficient pressure on the concrete of the previously cast blocks to compensate for fully one-half the temperature range expected at the base of the dam, and considerably more than the full temperature range in the upper, more flexible, part of the dam. The expander joints were concreted when the temperature was approximately at its minimum value. Temperatures were determined at three levels in the dam and at three points at each level, by means of thermocouples placed one at the middle and one each 1 ft. from the up-stream and down-stream faces of the dam, respectively. This dam was built elliptical in plan with increasing axes at the higher levels. The longer axis was parallel to the axis of the valley. In this way and by means of the expander joints an attempt was made to relieve the dam of part of the stresses due to contraction of the concrete, an idea which has since been developed in a more definite way by Mr. Karpov.

D. C. Henny,[8] M. Am. Soc. C. E. (by letter).—The clear and complete manner in which the results of research at Ariel Dam have been presented does great credit to the authors. The very thoroughness of the paper renders correction of any misstatements all the more desirable. Attention is called, therefore, to what was said as to sprinkling of the concrete, as follows, "* * * all surfaces of the arch, as well as those of the remainder of the dam, were sprinkled continuously with water" and again, elsewhere, "the outside surfaces exposed to the sun were kept wet constantly by sprinkling." These statements are correct only as to the advice given by the writer in his capacity of Consulting Engineer of both the State and the Northwestern Electric Company. He deemed undisturbed continuation of the setting process, early cooling, and prevention of shrinkage from drying for a period of at least two months, of great importance, especially in the present instance where the heat rise in the concrete was rather high (70°F) and where closure of the arch had to await cooling of the concrete. The facts, however, were that for about ten days the concrete surfaces were kept moist most of the time and thereafter they were sprinkled occasionally, although somewhat better results were obtained during the latter half of the work.

In a number of arch blocks slight checks and cracks developed where the dimensions were about 30 and 40 ft one way and 40 to 60 ft the other way. They occurred on all faces generally half-way between corners and were due apparently to continued heat rise in the interior and to simultaneous shrinkage of the outside resulting from drying and cooling. Investigation by hydraulic test showed that these fine cracks did not penetrate more than 3 or 4 ft and, in the course of time, their width diminished. On later examination these cracks seem to be entirely closed, and where moisture conditions were favorable they may, in fact, have become completely healed.

In the matter of uplift it was stated that "the maximum observed uplift between the rock and the base of the dam was less than 20% of the static head and this was within 5 ft of the up-stream face." An important question is to what extent this favorable condition was due to foundation drainage. On the whole it is believed that the job of securing a good foundation and of grouting was done so thoroughly that even without drainage the uplift pressure would have been relatively low. For a part of the work, however, this is not true. Reference is made to the abutment block next to the arch on the north side of the river which is shown in plan in Figs. 4 and 6. From these diagrams it may be noted that this block has an addition projecting into the reservoir, which was made necessary by the heavy radial shear transmitted by the arch. A branch of the main drainage gallery extends into this addition, in which six vertical 2-in. pipes rise from the foundation. They are about 10 ft from the up-stream face and their aggregate flow is approximately $\frac{2}{3}$ gal per min with reservoir full and the water level 130 ft above the top of the pipes. With reservoir water 40 ft above the top of the pipes during the period of first filling, a test was made by closing all the pipes when it was found that the pressure quickly rose to 80% of the reservoir head.

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[8] Cons. Engr., Portland, Ore.

The block projection in the gallery in which this pressure was observed, is exposed to increep of water at the foundation line on three of the four sides, which implies a favorable condition for the development of high uplift pressure. This experience illustrates two points, namely, that on the one hand thorough drainage is desirable and on the other that no absolute dependence should be placed on it in the design.

Mention was made in the paper of the method of making vertical cooling holes with rubber cores. These cores are quite effective in mass concrete and have the great advantage of keeping the holes clean and free of trash. However, it was found difficult to maintain the holes straight and in their exact position. When an attempt was made to use them in the 2 ft of slot-fill concrete great difficulty was found in pulling them up without disturbing adjacent concrete, and their use in this connection was abandoned. Incidentally, it may be said that the only seepage now (1933) visible on the downstream face of the dam (except that coming through three small diagonal cracks from 3 to 5 ft long near the abutments), occurs through the concrete slot fill. This seepage is small and consists mostly of discoloration of the concrete.

It was to avoid such seepage that artificial cooling in connection with reasonably slow construction was urged. Seepage of this kind had been observed in similar cases; it is evidently due to the fact that vertical contraction during cooling is prevented by friction against the sides of the old concrete thus forcing the formation of fine horizontal cracks where, at construction joints or other places, tension exceeds the tensile strength.

As regards both deflection and temperature measurements new data are constantly becoming available at this structure, and it is hoped that the authors in their closing discussion will include this additional information.

The paper gives well deserved credit to the Company engineers and superintendents, but inadvertently omits mention of D. W. Cole, M. Am. Soc. C. E., who was responsible for securing proper foundation, grouting, and many other important features of the work.

B. E. TORPEN,[9] M. AM. SOC. C. E. (by letter).—The authors are to be commended for their full presentation of the data on dam deflections and concrete temperatures. It is hoped that follow-up measurements will be made and published at intervals of a year or two. By co-operative work of this nature at several dams, their behavior will become known in time, and such information will be useful in future design.

As yet little is known of the internal behavior and stresses in dams. Research and tests are very necessary and each additional ray of light tends to brighten the hope that some day dams may be designed with the assurance and the knowledge that they will serve as planned. To-day, there is too much uncertainty to permit unity of opinion among designers. It will require time, tests, research, and data on existing dams to crystallize opinion into a common knowledge. In addition to recording deflections, every effort should be made to determine the internal stresses in the concrete due to water load, heat, water-soaking, and the time-factor influence.

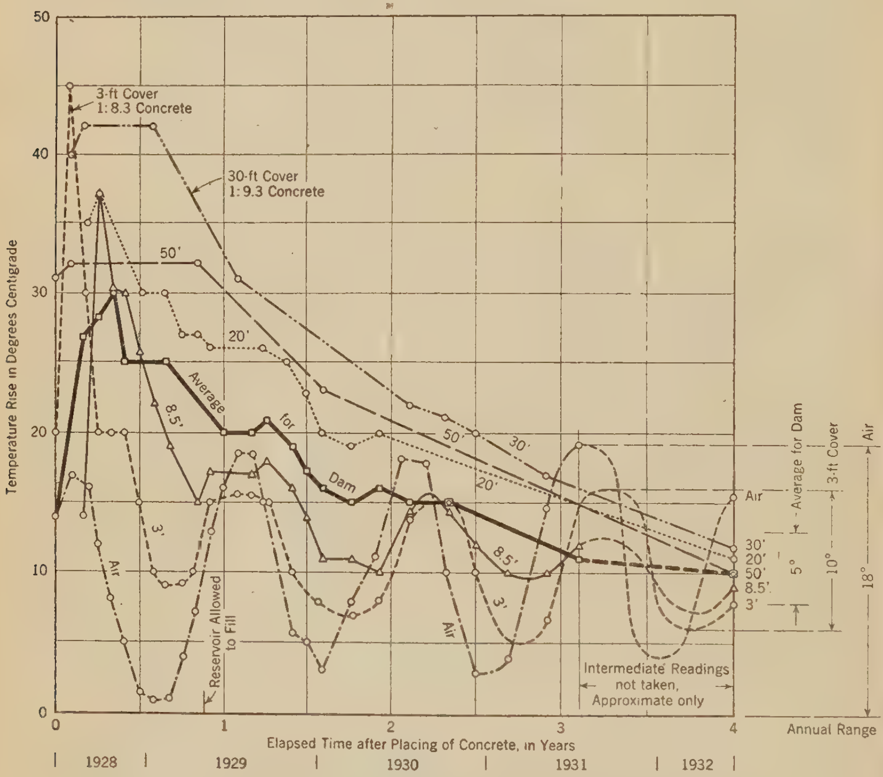

FIG. 25.—CONCRETE TEMPERATURES, BULL RUN DAM, PORTLAND, ORE.

The question of temperatures in setting concrete has long been of interest to engineers, and temperature readings have been observed since 1910, or thereabouts. At the Bull Run Dam, for Portland, Ore., fifty-two resistance thermometers and telemeters were buried in the mass of four 40-ft sections at various depths, and temperature readings were observed for several years. The results of these tests have been compiled[10] by Ben S. Morrow, Assoc. M. Am. Soc. C. E.

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[9] Engr., Corps of Engrs., U. S. A., Bonnesville Dam. Portland, Ore.

Temperatures at Bull Run Dam shown in Fig. 25 indicate that at 3 ft of cover, the concrete soon loses its chemical heat and follows the air temperature with a lag of about 5°C and one or two months in time. At 8.5 ft of cover, the cooling is slower and the effect of air temperature is less, but still quite marked. At 30 ft of cover, cooling is very slow, requiring about three years to return to the temperature at which it was placed.

The curve at 50 ft of cover is influenced by the proximity of foundation rock, and all curves are affected by water-soaking from the up-stream face, as the reservoir was allowed to fill in May, 1929. The "average" curve shows the influence of the fluctuating temperature of the exterior envelope.

At four years, the average annual range for the entire dam was only 5°C; at 3 ft of cover, it was 10°C, or about one-half the annual range of the air, which is 18°C.

The specific gravity of the aggregate at Ariel Dam was 2.38 as compared to 2.8 for aggregate at Bull Run (see Table 9).

TABLE 9.—COMPARISON OF AGGREGATE AND RESULTING CONCRETE

Item	Ariel Dam	Bull Run Dam
Specific gravity, gravel	2.38	2.8
Weight of concrete, dry, in pounds per cubic foot	138	154
Weight of concrete, wet, in pounds per cubic foot	147.5	157.9
Cement content, in barrels per cubic yard	1.00	0.88
Water cement ratio	1.05	0.97
Slump, in inches	1	3 to 4
Breaking Strength, in Pounds per Square Inch, at:		
28 days	2 800	2 100
6 months	3 750	....
1 year	....	3 500

Arch Site.—The Ariel Dam site does not appear especially favorable for an arch, but careful comparison of other designs showed this type to be the most economical in spite of the 2.3 to 1 ratio of span to height for the arch alone, and 3 to 1 ratio of span to height when the entire length of the dam site is considered. The variable radius feature of this arch dam has been in use only since 1912, but, since then, about fifty dams of this type have been constructed. As more data become available on arch deflections and stresses, this type of dam will become even more popular, due to its economy.

The authors have presented a mass of important data. The Ariel Dam is a beautiful structure, well built, and a fine example of modern engineering.

LARS R. JORGENSEN,[11] M. AM. SOC. C. E. (by letter).—The Ariel Dam is a fine addition to the arch dam family and is well described in this paper. The work it represents is fully appreciated.

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[10] Rept. of the Committee of Engineering Foundation on Arch Dam Investigation, May, 1933, Vol. III, Pt. III, p. 41.

[11] Cons. Hydro-Elec. Engr., Berkeley, Calif.

The cantilevers of the Ariel Dam have been calculated to take a certain part of the total water load, thereby relieving the arch for just that part. Accordingly, a dam with a slimmer cross-section has been obtained, than if the cantilevers had been left out of consideration for load-carrying purposes.

The general shape of the Ariel Dam, according to Fig. 4, remains as symmetrical as if the cantilever action had been neglected. In both cases, the up-stream and down-stream faces would be warped surfaces, bound by concentric circles at varying horizontal elevations, having lengths of up-stream radii decreasing toward lower elevations, similar to any ordinary constant-angle arch dam. Therefore, to all intents and purposes, the only difference between the actual design of Ariel Dam and one neglecting the cantilever action is, that the last-named would have greater thickness. With central angles the same in the two cases, the heavier section would have the smaller stress; thus, the problem finally simmers down to what the maximum stresses or the factor of safety ought to be.

Where a State commission or the Federal Power Commission dictates the maximum stress to be used in the design, as is the general rule almost everywhere, concrete can be saved perhaps by attempting to use the combined arch and cantilever method, but the same so-called saving, or the same thinner section, could have been obtained for the Ariel Dam by calculating the arch for the full load and increasing the arch stresses correspondingly. A small addition to the section near the abutments at the middle height of the dam would be of advantage to the design in any case.

The writer has visited the Ariel Dam and has never seen better workmanship. As far as the margin of safety is concerned it makes little difference whether the vertical cantilevers or the beams carry any load whatever. The arch stresses are kept low, according to data given in Tables 1 and 2, and the arch can safely carry much more load.

Only at the north abutment do the arch stresses given in Table 1 approach the allowable limit; but since the north abutment is only a continuation of the arch, except that the section of the continuation is about twice as large as that of the arch, the arch stresses determined under the assumption that the arch ends definitely at the thrust-block, are conservative.

The attempt to calculate the division of load between the cantilevers and the arches leads to tension in the up-stream face of the cantilevers, as it generally does unless the dam is quite high compared to its width. The authors do not accept this tension, but assume that cracks develop.

When a horizontal crack is produced it causes additional deflection of the cantilevers, which re-acts on the deflections of the arches and these, again, re-act on all the other cantilevers, thus causing a change in the division of load between cantilevers and arches. The writer has made a search of technical literature in order to ascertain how a deep horizontal crack, or several deep cracks, affects the deflection, since the ordinary theory is inadequate to deal with this problem. The only reference the writer has found[12] was one by Mr. B. F. Jakobsen, which deals with one crack only. This is a problem that is quite complicated and no claim is made to anything but a rough approximation.

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[12] "The Cracked Cantilever in Dam Design," *Engineering and Contracting*, December, 1930, p. 439.

The writer would like to know how the authors calculated the stresses and deflections produced by the cracks, and how they determined the location and depths of the cracks, because unless this can be done with some degree of accuracy the proposed method obviously fails or becomes a matter of pure guesswork. It would make considerable difference whether there were three cracks or five.

In Tables 1 and 2 the boundary stresses in the arches and in the cantilevers, respectively, are shown for a certain calculated division of load. For Cantilever *G* (Cantilever 27, Fig. 6), cracks are assumed to occur at various elevations. For intermediate elevations the depths of cracks are not given. To assume so many deep cracks is a rather astonishing practice. A crack 55.5 ft deep in a section 80 ft thick, leaving less than one-third the section intact, does not inspire any great confidence in the ability of the dam to act as a cantilever.

The writer has never seen such a variety of deep horizontal cracks in any dams, although he has spent much time in looking for them to study their cause, in order to avoid them in the future. He has found a number in various places in various dams, but never (or rarely) any definite horizontal cracks of any consequence. If they were present, such cracks would not be difficult to detect in dams provided with vertical drains near the up-stream face; they would also be indicated definitely with reservoir empty after they had closed. If as many horizontal cracks could be found on the up-stream face of the Ariel Dam as were assumed in Table 2, one would hesitate building the next dam.

The writer objects to a theory in which cracks are assumed as soon as apparent tension is encountered and in which the calculated depths of such assumed cracks are made the basis of deflection calculations and stress determinations. The stress distribution near such cracks has been disturbed to an unknown extent. One designer makes one guess, whereas some other equally competent engineer would make a different guess and would obtain different results. In the Stevenson Creek Dam, there was one horizontal crack at the bottom of the dam, but this structure is not of normal arch design and, therefore, not in the same class as the Ariel Dam.

On several arch dams, the writer has observed that the part of the arch between the quarter-points and the abutments, does not deflect in a down-stream direction as much as the deflection formula would indicate. An exaggerated case is the Stevenson Creek Dam, in which this part of the arch deflected up stream instead of down stream with full water load. This would indicate that the arch is loaded with more than the full water pressure toward the abutments. The deflection theory of arches as now available[13] does not take this phenomenon into account, as it cannot be explained theoretically, except perhaps as a buckling action.

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[13] "Etude sur les Barrages Arques," par Stucky.

The late Willis A. Slater, M. Am. Soc. C. E., stated[14] that on the Emigrant Creek Dam at Talent, Ore., "it was found that there were indications of an up-stream deflection" (of the arch toward the abutments). "With this double confirmation [Stevenson Creek Dam and celluloid model at Princeton University], there need no longer be any reason for doubting the indication of an up-stream deflection of the Emigrant Creek Dam." The Stevenson Creek Dam deflected 0.1 in, up stream toward the ends of the dam, with 60-ft head of water.

Ariel Dam has a crest thickness of nearly 20 ft and buckling would scarcely be expected. On this dam, however, both abutments moved radially, the south abutment 0.01 ft, which is twice that assumed in the design, and it is difficult to understand, why the deflection of Point *M*, Cantilever 36, Table 8, at Elevation 220 was only 0.008 ft (calculated to be 0.006 ft) unless some up-stream deflection of the arch at *M* relative to the abutment did take place, in which case the load on the cantilever is reversed. At Elevation 120, Point *M*, Cantilever 36, is located so close to the foundation and to the abutment that the arch could not buckle measurably up stream, and the measured deflection of 0.011 ft indicates that the foundation evidently has taken part in the down-stream deflection to a large extent.

The calculated deflection at this point, which is 0.003 ft, or $3\frac{2}{3}$ times smaller than the measured one, is supposed to create a horizontal crack 11 ft deep on the up-stream face. A much greater actual deformation would presumably cause a much greater depth of crack. It is realized that such small deflections are quite difficult to measure accurately, but such errors in calculating the deflection must produce great errors in the division of load between cantilevers and arches.

For finding the stress distribution on horizontal planes in a cantilever, three assumptions can be made (one at a time, of course): (1) The vertical compression on horizontal planes is a linear function of the ordinate, x; (2) shear distribution on horizontal planes is parabolic; or (3) vertical stress is distributed according to the principle of least work.[15]

For a triangular profile or its equivalent, Assumption (1) gives the usual trapezoidal distribution with a maximum compression at the down-stream face. Assumption (2) gives a large vertical tension at the up-stream face, a large compression in the middle of the cross-section, and zero compression at the down-stream face.[16] Assumption (3) gives less vertical compression (or actual tension) at the up-stream face and less compression at the down-stream face than Assumption (1). The stresses obtained by the use of this method will average between those due to Assumption (1) and Assumption (2), and since less important assumptions were made in obtaining the design formulas for Assumption (3), these formulas express most nearly the actual stress distribution on horizontal planes near the foundation. Assumption (1) gives stresses at the up-stream face that are too favorable.

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[14] Rept. of Committee of Engineering Foundation on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 130.

[15] "Stresses in Gravity Dams by the Principle of Least Work," by B. F. Jakobsen, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 489.

[16] "Technische Mechanik," by Otto Mohr, Second Edition, p. 296.

H. M. Westergaard, M. Am. Soc. C. E., has attacked the problem of arch design in a somewhat different manner.[17] He treats the structure as an elastic shell, rather than as a system of intersecting arches and cantilevers. Professor Westergaard advances the idea that when the influence of Poisson's ratio and of twisting moments is considered, the simple picture no more exists in which independent arches and cantilevers deflect equally at each point in common, when the load is divided properly between the arches and the cantilevers. He does not develop formulas, however, that can be used for calculating the stress, but states that the present stage of the work is that of research and is too elaborate to be used directly in design. A designing engineer cannot be expected to have the mathematical ability of Professor Westergaard and, therefore, would have to leave the problem of the shell dam alone.

The late Fred A. Noetzli, M. Am. Soc. C. E., developed approximate formulas for the vertical arch action in curved dams,[18] which tend to demonstrate that a considerable part of the weight (in the example given by Mr. Noetzli, it is one-half the weight) of the middle high portion of the dam is transferred by means of vertical arch action to the hillsides. Only the remaining weight could then take part in resisting the water load through cantilever action. This is an important item to consider and it is not safe to overlook it, as has been done.

When designing an arch dam using, say, the Cain formulas for determining the stresses and assuming full water load acting on the arch, the experienced designer will always keep in mind that certain actions not directly taken care of by the simple Cain formulas (such as the spreading of the foundation, a temperature drop, and water-soaking on the up-stream face) tend to increase the tension at the intrados at the crown. The action of Poisson's ratio, however, tends to compensate for this tension, but at higher elevations this compensation is small.

Since it is necessary, for architectural reasons, to produce a smooth symmetrical appearing structure, it is impossible to keep the stresses uniformly as great as the maximum, or anywhere near this limit, at all elevations. Only at elevations where rather high stresses exist, would it be necessary to go over the calculations repeatedly, taking each unfavorable item into consideration separately. Tensile stress at the intrados at the crown is the most important stress on which to keep a check.

For lowering a high tensile stress at the abutments on the up-stream side, there are several compensating features not included in the formulas, such as water-soaking, foundation deformation, etc. That the compression should occasionally over-run the present allowable limit of 600 lb per sq in. is of minor importance.

It is of advantage for improved distribution of arch stresses that the Ariel Dam is provided with contraction joints spaced rather closely together, as at each joint the stresses have an opportunity to adjust themselves somewhat.

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[17] Rept. of Committee of Engineering Foundation on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 231.

[18] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 36–39.

In the past the writer has followed a tendency to space contraction joints rather far apart; this was with the object of saving in cost of joints. It will lower the actual maximum stresses, however, to have the joints close together (say, 30 ft apart in the zones of maximum stress) and to grout them with a mixture of cement and plaster of Paris, with or without a combination of chemicals, depending on whether the cracks are large or small. Plaster of Paris grout, used alone, expands 0.3% when setting. The combination is nearly neutral in this respect; the modulus of elasticity is much smaller than for the cement grout alone and will allow for some re-adjustment of the arch stresses better than the pure cement grout.

The Ariel Dam section is thinner than it would have been, had the cantilever action been neglected. The same total yardage could have been obtained by increasing the arch stresses and neglecting the cantilevers as load carriers. The time saved in making such a design would equal the cost of many yards of concrete that could have gone into the structure instead.

As the technical theory stands developed to-day, with the necessity of making a considerable number of assumptions, in order to get anywhere it seems safest to depend on arch action only in arch dams or on gravity action only in gravity dams; but, even so, a combination of the two actions does occur, at least in some places.

H. B. MUCKLESTON,[19] M. AM. SOC. C. E. (by letter).—The writer has always felt some skepticism as to whether the results of computations by the trial-load method were worth the enormous labor involved. In the long run, the method is based on the elastic properties of the masonry and of the material in the foundations and abutments. It is certain that these properties are not uniform throughout the structure and, moreover, that they cannot be predicted with any certainty as to the facts. Hence, the behavior of the structure under load cannot be predicted with any certainty, either.

Tables 1 and 2 exhibit some predictions which are extraordinary, to say the least. For example, they predict cracks at Elevation 0, which are 56 ft deep extending two-thirds the thickness of the dam. The writer cannot accept these predictions as at all likely to be borne out by the facts. Is it a fact that the dam is cracked as the theory shows it should be? If not the theory is worthless. In Table 10 the writer has re-arranged Table 8. It will be seen that prophecy and performance do not agree by as much as 370

TABLE 10.—VALUES OF THE RATIO, $\dfrac{\text{MEASURED DEFLECTION}}{\text{CALCULATED DEFLECTION}}$

Elevation	POINT MEASURED (SEE FIG. 6 (*b*))							
	T	*A*	*C*	*E*	*G*	*I*	*K*	*M*
200.00	1.50	1.26	0.99	0.66	0.74	1.38	1.21	1.33
180.00	1.80	......	0.86	0.71	0.86	1.36	3.00	3.70
120.00	0.26	......	1.47	1.06	1.15	1.50	......	......
90.00	......	......	......	1.20	......	......	......	......

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[19] Cons. Engr., Vancouver, B. C., Canada.

per cent. What is the use of carrying calculations to the fourth decimal place if facts may differ from predictions by any such amount as this? Even at Elevation 120.00, differences of 150% are shown.

If calculated strains may differ from actual strains by 150%, how far apart are the corresponding stresses? The theory indicates that either very high tension must exist in the up-stream face or, in the alternative, that cracks 56 ft deep have formed. This tension, or crack, is deduced from the deflection, as calculated, in the vertical plane. If a swelling of the masonry is assumed of only 1 part in 20 000, due to soaking, it will account for enough deflection to indicate nearly 100 lb per sq in. Poisson's ratio, assumed at 0.10, will account for nearly 50, the modulus being assumed at 2 000 000. If, as is much more likely, this modulus approaches 5 000 000, these two factors will account for considerably more.

If prophecy and performance differ by as much as this paper indicates they may, there is evidently something wrong with the theory upon which the prophecy is made.

IVAN E. HOUK,[20] M. AM. SOC. C. E. (by letter).—The experimental measurements at Ariel Dam show three facts: First, that internal temperature problems are important in the design and construction of concrete dams; second, that effects of internal temperature changes can be controlled adequately by proper precautions in planning and building large concrete structures; and, third, that the amplified trial-load method of analyzing arched masonry dams, including proper allowances for effects of twist, tangential shear, and foundation and abutment deformations, furnishes a satisfactory basis for the design of such structures.

Temperature changes in the interior of concrete structures have been measured and reported in engineering literature many times during the last thirty years. As far as the writer can determine, the investigations made by Thaddeus Merriman, M. Am. Soc. C. E., at the Boonton Dam, in New Jersey, during the period from 1903 to 1906, constituted the first real attempt to measure and analyze temperature changes in the interior of a large masonry dam.[21] This was mentioned in a résumé of all available concrete temperature observations prepared by the writer in 1930 and 1931. In two papers[22] the writer included tabulations of maximum concrete temperatures, maximum concrete temperature rise caused by generation of chemical heat during the setting period, maximum range in concrete temperature caused by seasonal variations in air and water temperatures at the faces of the structures, and pertinent descriptive data, with discussions of certain concrete temperature investigations being conducted by the U. S. Bureau of Reclamation at that time.

Various additional data on concrete temperatures have been published in foreign engineering periodicals, and in the Final Report of the Engineering

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[20] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

[21] *Transactions*, Am. Soc. C. E., Vol. LXI (December, 1908), pp. 413–429.

[22] "Temperature Variations in Concrete Dams," by Ivan E. Houk, *Western Construction News*, December 10, 1930, pp. 601–608; also, "Setting Heat and Concrete Temperatures," by Ivan E. Houk, *Western Construction News*, August 10, 1931, pp. 411–415.

Foundation Arch Dam Committee,[23] since the preparation of the aforementioned tabulations. However, the paper on the Ariel Dam investigations furnishes the first comprehensive treatment of the subject in all its varied relations to the design and construction of a large arch dam.

The authors gave a value of 143.4° F for the maximum concrete temperature observed at Ariel Dam, and a value of 74° F for the maximum placing temperature; but they do not state whether these measurements apply to the same location in the dam, or whether they can be used in calculating the maximum rise in concrete temperature caused by generation of chemical heat during the curing period. The writer would like to know what the maximum temperature rise was in any part of the structure and where, and under what conditions, it occurred?

Considering the dimensions and comparatively rapid rate of construction, it is probable that the resistance thermometers placed near the center of Ariel Dam were in a condition of practically perfect insulation from the time of installation until the chemical actions accompanying the curing of the concrete were completed. Consequently, the maximum rise in concrete temperature shown by these thermometers would be a measure of the maximum rise to be expected in a concrete structure of any size, assuming it to be built with the same brand of cement, the same kind of aggregate, and the same proportions of concrete ingredients used at Ariel.

In preparing the aforementioned résumé of available concrete temperature data the writer found that the maximum rise measured in a large masonry dam up to that time, reduced to a common basis of 1 bbl of cement to 1 cu yd of masonry, occurred at the Bull Run Dam, near Portland, Ore., and amounted to 63.4° F. Incidentally, it is understood that laboratory tests of concrete specimens from the Bull Run and Ariel Dams showed very similar thermal properties. However, the temperature data for Owyhee Dam, have been revised[24] on the basis of later concrete temperature measurements and show somewhat higher values. The maximum temperature of 120° F, and the maximum temperature rise of 50° F, given in the original compilation should be increased to 125.3° F, and 65.9° F, respectively; so that the maximum rise observed in Owyhee Dam was slightly higher than that at Bull Run Dam.

Considerably higher maximum temperatures and maximum temperature rises due to setting heat have been observed in structures built with richer concrete mixtures, as, for instance, in tunnel linings and large building foundations. A maximum rise of 100° F was observed in the construction of the strong-room of the Bank of England a few years ago.[25]

In studying the variations in lake temperature, shown on Figs. 18 and 20, it would be desirable to know what changes in reservoir surface elevation occurred during the periods of time represented by the curves, since seasonal variations in water temperature are dependent, to a large extent, on the depths of water above the elevations of temperature measurement. Measurements of the temperature of reservoir water at different depths are important because of its direct effect on the temperature of the concrete near the up-stream face of the dam.

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[23] Engineering Foundation Arch Dam Investigation, Rept. by Committee, Vol. III, The Engineering Foundation, May, 1933.

[24] *Western Construction News*, August 10, 1931, Tables 1 and 2, pp. 413–414.

[25] "Temperature and Humidity Effect on Concrete Defined." *Concrete*, June, 1929, p. 52; abstract of a Lecture before the Royal Inst. of British Archts., in London, England, by Oscar Faber.

The writer would like to know what kind of resistance thermometers were installed in Ariel Dam? what kind of cables were used in extending the electrical connections to the switchboards? and whether the leads within the masonry were placed in metal conduits? Inadequate design of thermometers, thermometer lead connections, and thermometer lead insulation constituted one of the principal sources of trouble in connection with early investigations of concrete temperature variations. It was not at all unusual for from 25 to 50% of the total number of thermometers in a single structure to become defective in some of the earlier investigations due to such causes, long before a satisfactory series of temperature measurements was secured.

Unfortunately, resistance thermometers made by the manufacturers of scientific instruments, such as were used in many of the early measurements of concrete temperature, were designed primarily with the idea of providing a high degree of precision in the observations rather than continuous satisfactory service through long periods of time under the conditions existing in such structures. Resistance thermometers installed in the interior of a masonry dam need not be read closer than the nearest degree. However, they do need to be capable of indicating actual concrete temperatures within 1° of the true values continuously during a period of time lasting several years after the completion of the dam.

The resistance thermometer used in the concrete temperature investigations at Gibson Dam, a 200-ft concrete arch dam constructed on the Sun River Project, in Montana,[26] a few years ago, was a substantial, home-made instrument, consisting of an insulated resistance coil placed in an 8-in. sheath of standard $1\frac{1}{4}$-in. galvanized pipe. The pipe was capped at both ends, and a hole provided in the center of one cap to serve as an exit for the armored cable leads. The resistance coil was held firmly in place inside the sheath, and adequately water-proofed, by pouring the pipe full of pothead compound through a small hole near the exit of the leads. The instrument was placed in freshly deposited concrete, and the leads laid in a shallow trench excavated at the surface of the lift before the concrete hardened, no metal conduit protection being provided.

Similar, but slightly modified, home-made thermometers were installed in Owyhee Dam, a 421-ft, curved, gravity concrete dam recently completed on the Owyhee Project, in Eastern Oregon. The modification consisted of lengthening the resistance coil so as to reduce the heating of the instrument during readings, providing No. 16 A. W. G. rubber insulated, fixture wire leads, 12 in. long, and placing a tee-and-nipple splicing chamber at the end of the thermometer sheath, so that the 12-in. leads could be properly connected to the main cable lengths at the time of installation. Lead-covered, armored cable leads were used on 113 of the 127 thermometers installed for general temperature observations, and special types of rubber-insulated cable on the remaining 14, no metal conduits being used for any of the leads. Three instead of two conductor cables were used, so that the resistance of the leads could be measured and deducted from the total resistance of the thermometer circuit each time a reading is made.

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[26] *Western Construction News*, December 10, 1930, p. 604.

Thus far, only 3 of the 31 thermometers installed in Gibson Dam, and only 6 of the 113 armored, cable-connected instruments installed at Owyhee Dam have become defective. The thermometers at the Gibson Dam have been in place a little more than five years, and those at the Owyhee Dam about two years.

Some engineers have objected to the use of lead-covered, armored cable for leads on resistance thermometers embedded in concrete, because of the possibility of damage during installation and the tendency of the calcium hydroxide in the green concrete to corrode the lead covering during the early part of the curing period. One engineer predicted that all the Gibson Dam thermometers would be out of order within a year, due to the corrosive action alone. However, the record of the Gibson Dam shows that the detrimental effects of the calcium hydroxide on the lead sheath are of minor importance as compared with the advantage of securing absolute watertightness during the early stages of installation. Apparently, the lead covering furnishes an additional protection against damage of insulation by workmen, or by moisture penetration, at the time such protection is most needed. The different types of improved rubber insulated cables being tried in the Owyhee Dam installations have not been in place long enough to furnish accurate comparative data. Rubber insulated cables, placed in well-drained, metal conduits, may prove adequate for long-time measurements. Nevertheless, from the standpoint of securing satisfactory data on concrete temperatures through a period of several years, the writer prefers rubber insulated, lead-covered, armored cable leads, suitably protected at contraction joint crossings.

The writer would like to know if any cores were obtained in blocks artificially cooled; and, if so, how the strength of such cores compared with the strength of those drilled in other sections of the dam? He would also like to know if any strength tests were made on the artificially cooled experimental block mentioned in Item 1 under the heading, "General Observations," and what average results were obtained if such tests were made? In other words, did the increased rate of heat dissipation due to artificial cooling cause beneficial or detrimental effects on the strength of the finished product; and, if so, what were the nature and extent of such effects? Obviously, the resultant effects on the strength and safety of the structure are the most important matters to be considered in determining whether or not refrigeration of concrete should be resorted to in building a large masonry dam. For instance, such effects are much more important than the advantages or disadvantages of the method from the construction standpoint.

As regards the comparison of calculated and measured deflections in the arch section of the dam, immediately following Fig. 21, it might be pointed out that an additional reason for the difference in deflections at the center of the arch is the absence of uplift pressure in the cantilever elements noted in Item 6, under "General Observations." The design assumption of uplift pressure over one-half the horizontal area of the base, together with the assumption of cracking in the tension areas of the vertical sections, as mentioned under "Design of Arch Dam," would naturally result in an excess of calculated deflections over measured deflections. This would mean that the cantilever elements near the center of the dam are carrying more load, and the arch elements less load, than was calculated in the trial-load analysis. It would also mean that the actual maximum stresses in the loaded dam, in both arch and cantilever elements, are somewhat smaller than those determined by the trial-load analysis in the latter part of "Design of Arch Dam."

The fact that the measured abutment movements at Ariel Dam are somewhat greater than the deflections calculated on the basis of the theory of elasticity, may be partly due to the opening of cracks, seams, or fissures in the natural rock formations. The writer has never been satisfied that natural canyon walls can be considered as homogeneous elastic bodies and that their movements can be determined accurately by a mathematical treatment of the various resultant forces.

The deflection measurements at Ariel Dam, considered as a whole, constitute a satisfactory check on the design methods. There are always many uncertainties involved in attempting to represent actual field conditions at a large masonry structure by definite mathematical formulas. Nevertheless, the Ariel measurements show that such representations can be made with a reasonable degree of confidence. They confirm the conclusion previously reached by the writer as a result of his work on models of arch dams and on technical studies connected with the design of Boulder Dam—namely, that the amplified trial-load method of analysis, making proper allowances for twist, tangential shear, foundation deformations, and other important effects—can be used safely as a basis for the design of important arch or curved gravity dams.

A. V. KARPOV[27] AND R. L. TEMPLIN,[28] MEMBERS, AM. SOC. C. E. (by letter).—This paper is of great value and is indicative of the modern trend to base the design of dams not on arbitrary theoretical assumptions, but on data that have been checked and proved by actual tests. As far as the temperature measurements are concerned, the paper discloses a thorough experimental study. However, the deflection measurements were made with less thoroughness and, unfortunately, strain or stress measurements were omitted.

For a comprehensive study of the behavior of a dam, the temperature measurements as well as the deflection measurements are important and necessary, but they can hardly be considered as the final step in an investigation of this kind.

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[27] Designing Engr., Hydr. Dept., Aluminum Co. of America, Pittsburgh, Pa.

[28] Chf. Engr. of Tests, Aluminum Co. of America, New Kensington, Pa.

From the modern engineering point of view, the determination of stresses is the final goal of any experimental structural testing program, and all measurements are of value only as intermediate steps leading to that final goal.

Deflection Measurements.—There does not seem to be a generally accepted method of deflection measurements. Three methods were used in the past: (1) In 1926 the clinometer was used to measure the deflection of the Stevenson Creek Dam[29]; (2) between 1921 and 1928 the deflections of a number of dams in Switzerland were determined by precise triangulation[30]; and (3) in 1930 and 1931 the deflections of the Calderwood Dam were determined by direct measurements, using a special tape, and measuring the change of distances from a central pier.[31]

Every one of these methods, if properly applied, gave satisfactory results as far as the relatively large radial deflections are concerned. None of them can be considered as giving satisfactory results if the small tangential deflections are to be determined exactly. The first and third methods require that the down-stream face of the dam be made accessible during the deflection measurements. The second method can be applied without having access to the down-stream face of the dam during the tests.

The writers studied the possibility of applying all three methods, and came to the conclusion that the triangulation method has advantages when only a few measuring stations are to be used. Measuring the deflections at a large number of stations requires considerable work to evaluate the data and, consequently, the results of the tests are available only after an appreciable time has elapsed. This method has one distinctive advantage; it permits an easy study of the foundation and ground movements.

The clinometer has considerable advantages if measurements are made on a large number of stations; but foundation and ground movements can scarcely be studied by this method, unless some additional provisions are made.

Direct measurements with a tape and by the use of a central pier represent a reasonable method which makes it possible to measure deflections at a comparatively large number of stations; and, at the same time, a study of the ground movements is possible, particularly if more than one pier is used.

There seems to be a tendency to prefer the triangulation method, but the writers believe that, in many instances, this preference is based rather on an insufficient appreciation of the other two methods, than on the advantages of the triangulation method.

Deflection Measurements Made in the Ariel Dam.—One of the most recent and very important advances in the arch dam theory is the realization that such a dam cannot be considered independently, but that it is an integral part of the surrounding canyon. Consequently, it is impossible to draw a sharp division line between the behavior of the structure proper and the ground that supports it.

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[29] *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3.

[30] "Deformationsmessungen an Staumauern," von W. Lang, Verlab der Abteilung für Landestopographie Bern, 1929.

[31] *Proceedings*, Am. Soc. C. E., December, 1933, p. 1565.

In making an experimental study of a dam the behavior of the ground is as important as that of the dam proper and it is hardly possible to arrive at satisfactory conclusions unless the behavior of the ground is disclosed. It seems to the writers that in the measurements at Ariel Dam the ground conditions were neglected somewhat; at least, the published data and diagrams do not give sufficient information about the deflections of the ground.

As far as the authors' conclusions, based on deflection measurements, are concerned, the writers cannot concur with a number of statements: (1) The authors state that the agreement reached on the Ariel Dam can be characterized as "entirely satisfactory" (Conclusion 8). Not only is there a considerable difference in the values of deflections, but even the character of the computed and measured deflection curves is different; (2) in Item 3 under "Deflections of the Arch Dam" the authors state that "from the maximum point the deflections taper off smoothly to the abutments and foundations." The July, 1932, measurements not only do not support, but rather contradict, that statement. The deflections at Elevations 120 and 180, as well as at the vertical section on the *E* line, clearly show this contradiction; (3) in Item 7, under "Deflections of the Arch Dam," the authors make a statement that the arch action took place as expected, basing their conclusion on the close agreement between measured and calculated tangential displacements. It is difficult for the writers to agree, considering the large discrepancies between the measured and computed radial deflections. The tangential displacements are small, and these measurements are, of course, much less reliable than those of the radial deflections. Considering both the radial deflections and tangential displacements, it would seem that the arch action was quite different than was expected.

Considering the radial deflections, it would appear that, as far as can be judged from Fig. 21, the agreement, in a majority of cases, between the measured and computed deflections, when adjusted only radially, is closer than between the measured and computed deflections when adjusted finally. This would seem to indicate that either the design assumptions or design method, or both, are incorrect. Furthermore, it would seem that the disagreement between computed and measured deflections would increase if the design was changed so as to take care of the actual deflections (which were larger than originally assumed) of the concrete thrust-block and of the south abutment (see Items (5) and (6), under "Deflections of the Arch Dam").

Determination of Stresses.—The paper does not disclose the magnitude of the actual stresses in the Ariel Dam, but the reader is left under the general impression that since there seems to be a fairly close agreement between the computed deflections and those actually measured, an agreement between the computed and actual stress should be expected. This is a surmise which is not proved either in the Ariel Dam, or by any other commercial dam. Until that gap is bridged, there can be no assurance that a design is safe, rational, and economical.

In general, it is much easier to arrive at a close agreement between the computed and actual deflections than between computed and actual stresses. Examining the measurements described in the paper from this point of view, the writers believe that the results of the deflection measurements rather indicate that there is a very considerable difference between the computed and actual stresses in Ariel Dam. While the tests described by the authors are quite in conformity with the published statement of the Engineering Foundation Committee on Arch Dam Investigation,[32] yet the results obtained by use of different methods in the tests conducted by the writers would appear to substantiate the comments made in this discussion.

Trial-Load Method.—The Ariel Dam was designed painstakingly, using the trial-load method, and according to the best standards accepted in connection with this method. Realizing the fact that the customary assumption of linear distribution of stress is far from being correct, an additional refinement was attempted. In order to compensate for the non-linear distribution of stress, the very doubtful mean of assuming an entirely arbitrary high modulus of elasticity in shear was used, as seems to be customary in the latest designs made by the trial-load method.

The tests described in the paper are of great interest and show that although the trial-load method may be an advance over design methods of the past, nevertheless, it apparently still admits of considerable improvement. Probably the most important improvement lies in the shape of the horizontal arches.

At present, it is generally admitted that notwithstanding the fact that the hydrostatic water pressure is constant at every horizontal elevation, the load imposed on a horizontal arch varies, a division of the total load between the arch and the vertical elements taking part. In spite of that knowledge, the arch dam is one of the few modern structures subjected to hydrostatic pressure in which the shape of the structure is arbitrarily chosen. The circular shape of the horizontal arches is about the same as if an attempt should be made to build non-circular large-sized tubing to be subjected to a uniform hydrostatic pressure; this would result not only in poor utilization of material, but the exact evaluation of stresses in such a tube would be of extreme complexity. A dam composed of circular horizontal arches is quite similar, but instead of a non-circular tube subjected to uniform hydrostatic load, a circular arch is subjected to non-uniform load.

Next, it is necessary to recognize the fact that a dam cannot be built as a monolithic structure, and, consequently, a design based on the assumption of such a structure cannot represent the actual conditions. The trial-load method simply neglects the existence of the vertical joints in spite of the fact that such joints influence the stress distribution considerably. Such a method results in considerable difference between the actual and the design stresses. One advance in arch dam design (that is excluded by the present trial-load method) will be made possible by the clear recognition of the fact that if the dam is built by current methods, the vertical joints are inevitable, but may be arranged so as to be useful.[33]

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[32] Arch Dam Investigation," p. 6, Vol. III, May, 1933, pub. by Engineering Foundation.

Finally, the close correlation between the behavior of the ground and of the dam proper is to be taken into consideration. This correlation is partly expressed in the advantages of avoiding sharp changes in conditions at places where the dam proper meets the foundations or abutments. The consequent liberal use of properly designed fillets is again an important design improvement that would be made clear if, instead of the linear distribution of stress and high modulus of elasticity in shear, some more scientific assumptions were introduced.

The usual trial-load method requires considerable time and effort, but neglects entirely, or partly, these important considerations. Arch dams are very responsible and expensive structures, and the expenditure of time and effort in their design is fully justified, provided the evaluated stresses are reasonably close to those obtained in the actual structure, assuring the safety and economy of the design. As the measurements on Ariel Dam clearly show, in spite of the painstaking design, a considerable difference appears between the actual conditions and the conditions computed by the use of this method.

The increase of engineering knowledge in design of dams in general and arch dams in particular is of extreme importance. The data presented in the paper show that the engineering advance made in the past few years is not sufficient. Much more theoretical and research work is necessary to enable a dam to be designed on the basis of sound engineering knowledge, as is imperative for structures of that size and responsibility.

The foregoing criticisms do not detract in any way from the value of the paper, and are made with the idea of indicating that the design of arch dams has reached a point when further advance depends on measurements of a more complicated nature than were attempted in Ariel Dam.

A. T. LARNED[34] and W. S. MERRILL,[35] MEMBERS, AM. SOC. C. E. (by letter).—In submitting their paper the writers anticipated that it would be possible, finally, to present data extending the temperature and deflection records to a time when practically all the heat of hydration had disappeared from the arch concrete. Fig. 18 shows the record of thirteen thermometers in Block 22 (artificially cooled) at Elevations 100 and 150 from February, 1931, to February, 1932. Fig. 26 shows the record for Elevation 100 extended to March, 1934. These data show nothing startlingly new or different, but confirm the results expected. The principal facts shown by these temperature records are that: (1) The temperature of the concrete follows the seasonal variations of the air and water temperatures, but lags considerably behind

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[33] See "The Compensated Arch Dam," by A. V. Karpov, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 1309.

[34] Engr., Elec. Bond and Share Co., New York, N. Y.

[35] Hydr. Engr., Elec. Bond and Share Co., New York, N. Y.

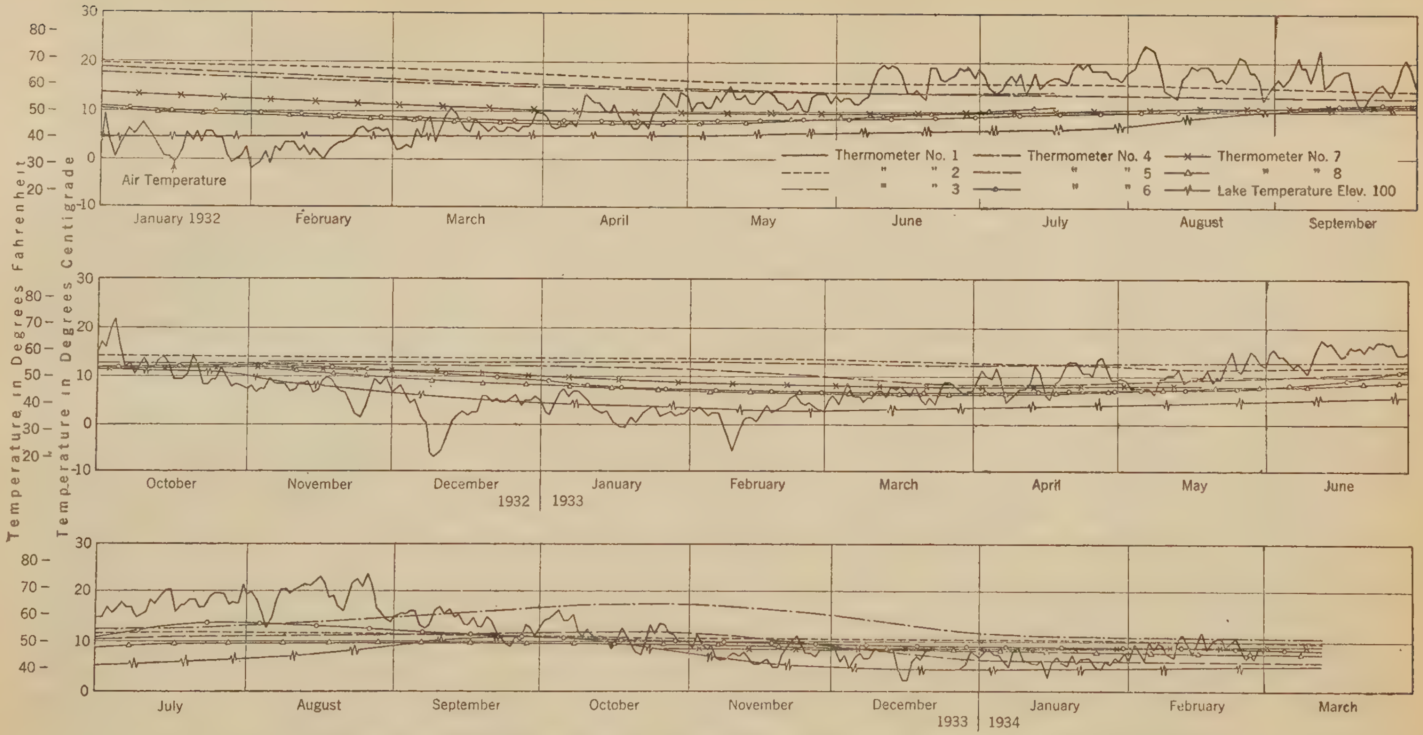

FIG. 26.—TEMPERATURE RECORDS FOR BLOCK 22 (ARTIFICIALLY COOLED); CONTINUATION OF FIG. 18 (*b*)

them, especially in the interior of the mass; and (2) the temperature of the concrete is slowly but gradually decreasing. For much of the arch the temperature of the concrete has already reached the mean annual air temperature of 12° C. The thicker portions of the structure are still above the mean temperature, however.

The deflections of the arch were shown in the paper up to July 15, 1932, the base line or zero point being the measurements made on May 13, 1931. The continuation of these measurements an additional eighteen months (complete records for two points only are shown in Table 11), shows the variation of the deflections with the changing temperature of the air and water and indicates some increase in the maximum. In general, the deflections at the higher elevations are less than those computed. Professor Davis commented on this fact in connection with the previous measurements and stated that this probably could be accounted for by assuming that the modulus of elasticity of the concrete was actually much higher than the value of $E_c = 2\,000\,000$ lb per sq in., which was used in the design.

The writers also commented on these smaller deflections and stated that the foregoing consideration might be one of the reasons why the arch deflected less than was calculated. Other reasons advanced as the possible cause of the smaller deflections were the fact that the calculated values were for water at Elevation 240, whereas the measured values were for water at elevations somewhat less. The third reason given was that the concrete had not yet reached the design temperature, consequently, the deflections would not yet have reached the maximum. There would seem to be little doubt that the concrete has a higher modulus of elasticity than that computed and that the arches are stiffer than calculated.

The probability that the uplift under the arch is less than that assumed in the design, and the possibility that the cantilevers did not crack to the extent calculated, are addditional reasons for the smaller arch deflections.

Table 11 shows the deflections of Points *E*1 and *E*2 (Fig. 6), at Elevations 220 and 180 near the crown, from the beginning of the record to January, 1934. These points have been selected as representing the most flexible part of the arch. It will be noted that the deflections increase gradually up to May, 1932, the loss of heat from the concrete during the summer of 1931 exceeding that given to it during the hot months. Following this, the deflections de-

TABLE 11.—RADIAL DEFLECTIONS OF ARCH, IN FEET

Date	*E* 1, at Elevation 220	*E* 2, at Elevation 180	Reservoir elevation	Date	*E* 1, at Elevation 220	*E* 2, at Elevation 180	Reservoir elevation
July, 1931	0.019	0.028	201.5	July, 1932	0.095	0.090	236.0
August, 1931	0.023	0.035	215.0	September, 1932	0.073	0.075	225.7
October, 1931	0.035	0.038	213.0	October, 1932	0.050	0.054	203.4
November, 1931	0.057	0.075	230.0	January, 1933	0.110	0.102	233.4
January, 1932	0.078	0.085	231.5	April, 1933	0.100	0.099	230.0
March, 1932	0.092	0.089	230.0	July, 1933	0.105	0.099	236.7
May, 1932	0.108	0.100	234.4	October, 1933	0.100	0.100	235.0
				January, 1934	0.115	0.104	234.0
Computed	....	....	....	Computed	**0.164**	**0.140**	**240.0**

crease and increase regularly with hotter and cooler weather, respectively—as they should. Comparing January, 1932, with the same months in 1933 and 1934, it is seen that the deflection has increased materially. In all cases the deflections lag somewhat behind the air and water temperatures.

The size of the slots between the blocks, of course, has a considerable influence on the final maximum deflection of the arch, and it is generally agreed that they should be kept as small as possible in order to minimize the amount of setting heat to be removed from the concrete therein. In the arch dam at Waterville, N. C., the slots were made 8 ft long, with the blocks 48 ft long. The 2-ft slots in the Ariel Dam were made as small as it was believed feasible to fill properly. The type of slot shown by Professor Davis might be feasible in a small dam with a total thickness of only 18 ft, but would be difficult to fill in a dam such as Ariel, having a total thickness of as much as 90 ft and a height of more than 300 ft.

It is true, as mentioned by Mr. Henny, that the surface of the arch concrete was not kept as continuously wet as anticipated. It is believed, however, that the moderate deficiency in this respect was not sufficient to allow the concrete to dry out appreciably. As stated by Mr. Henny, the number and depth of cracks in the arch were comparatively few and shallow; finally, they closed almost completely, so that the arch is now (1934) remarkably free from cracks of any kind.

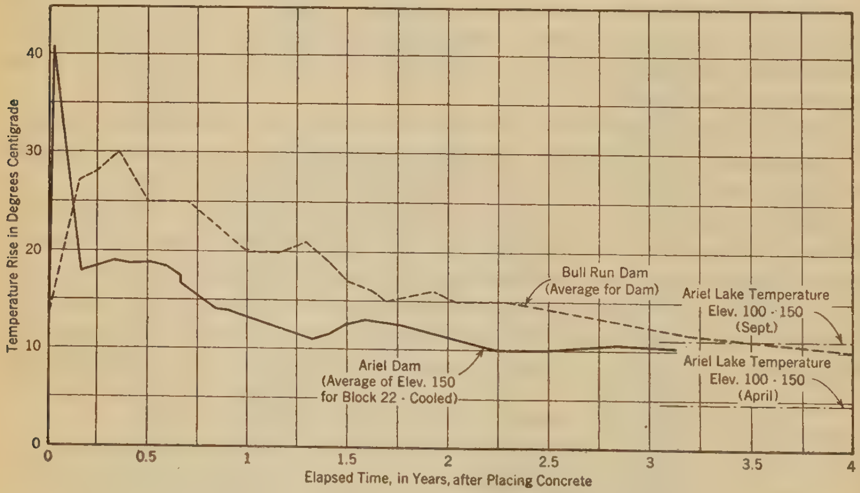

FIG. 27.—COMPARISON OF CONCRETE TEMPERATURES.

The data contributed by Mr. Torpen for the Bull Run Dam are of great interest, especially Fig. 25, showing the temperature at different distances in from the face of the concrete and also the average temperature for the entire dam. The writers have made up Fig. 27 to show how the average temperature of Block 22 (artificially cooled) has varied as compared with the average of the Bull Run Dam, which was constructed in the conventional way with only joints between the blocks. It will be seen that the maximum temperatures (143.4° F) for the concrete in Ariel Dam were considerably higher than those at Bull Run Dam, but that the decrease in temperature was considerably more rapid, reflecting the influence of the artificial cooling of the concrete. It is worthy of repetition that artificial cooling of concrete is being utilized on a large scale at Boulder Dam. The adoption of this program for cooling has made possible the fast construction schedule now being realized on that job.

Mr. Jorgensen has questioned the desirability of going to the trouble of designing an arch dam by the "trial-load" method. It is true that it may be done more simply and quickly by the Cain method. He admits, however, that the cantilevers do carry a part of the load, thereby allowing a thinner section to be secured by the "trial-load" method. He suggests that the thicker arch, as designed by the Cain method, would have lower stresses, and that, therefore, it would be permissible to adopt a higher stress for the Cain method, in order to keep the section down to the same thickness as determined by the trial-load method. The writers believe that if a higher stress can be used for the concrete in one structure the same stress should be acceptable in all similar structures. Previous to designing the Ariel Dam the writers had experience in the design of the Waterville Dam by the Cain method of analysis. In this case, as in that of many other arch dams designed by this method, the maximum compressive stress in the concrete was computed as 600 lb per sq in.; and this was the value used for the Ariel Dam, also.

It is quite possible, and perhaps probable, that the cracks in the cantilever elements did not develop to the number and depth indicated. Additional stiffness of the cantilevers, due to more shallow, or a smaller number of, cracks may be one of the reasons why the deflection of the structure was less than calculated. Mr. Jorgensen indicates his belief that one designer would make one assumption as to the number and location of cracks, while another would make entirely different assumptions, thereby getting a considerable variation in results. Such a variation is not possible, however, because the number, location, and depth of the cracks are the direct results of the application of the theory itself.

The writers have already stated their belief that the "trial-load" method of analysis for arch dam design is the best one yet developed. It does require a large amount of work, but the results are worth while. The application of the theory is constantly being made simpler and easier. Considerable simplification of the theory and additions to it were made in the design of the Ariel Dam. This work was carried out by designers of exceptional mathematical ability. The theory of the trial-load method of analysis has been discussed extensively in technical literature, and several references to it were made in the paper. In another paper,[36] Mr. Houk has discussed this subject further, referring again to the tests of models that have confirmed the results of calculations in several cases.

Mr. Muckleston has compared the measured deflections of the Ariel Dam with the calculated deflections and has shown that the results are a considerable distance apart in several cases. It cannot be denied that the two sets of results do vary considerably in percentage, but it must be remembered that the actual measurements are extremely small and, although the measurements were carefully made, extremely accurate results cannot be expected. Measurements could not be made on a laboratory basis, and it may be that some of them are in error. It is doubtless true, however, that some of the discrepancies are due to the fact that the structure is stiffer than calculated and that it does not act in all respects as calculated. The consensus of opinion seems to be that the measured results, in general, are sufficiently near the calculated results to indicate general agreement with the theory. It should be noted in this connection that stresses do not vary directly with deflection.

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[36] *Western Construction News and Highway Builder*, November, 1933.

Mr. Houk has inquired as to the characteristics of the resistance thermometers and the method of placing them and their leads in the concrete, and as to the maximum concrete temperatures observed in the dam. He also asks regarding the location of the cores removed from the dam. The resistance thermometers were designed by R. W. Carlson, Assoc. M. Am. Soc. C. E. They consist of fine, enameled, copper wire wound on a fiber core and covered with bakelite enamel and black asphalt paint. The leads consist of two No. 10 copper wires thoroughly insulated and protected with a cloth coating. The coils have a resistance of 12.5 ohms. These instruments were placed directly on top of a concrete lift and were firmly secured to a steel anchor, thus preventing movement. Where thermometers were placed in some of the heavy, continuous batches of concrete they were anchored to long rods driven well down into the fresh concrete. The leads were protected by conduit in places where they were exposed vertically or diagonally during the pouring of the concrete. The resistances are read at nine or ten small terminal boards around the dam. A testing set with a Wheatstone bridge is used. Allowance is made in all cases for the resistance of the leads and of the portable clamp on the connections to the test set.

The thermometers mentioned by Mr. Houk are doubtless more substantial than those used at Ariel Dam and the protection given them is better. Nevertheless, the Ariel thermometers are substantial enough and well enough protected so that a very large percentage of them are still in good operating condition.

The maximum concrete temperature of 143.4° F observed at the Ariel Dam occurred in Block 32 at Elevation 160, 85 ft above the base. This block was begun on July 27, 1930, in very hot weather, and concrete was placed rapidly in 10-ft lifts from Elevation 75 to Elevation 210, which was reached on August 16, 1930. The maximum placing temperature of 74° F, also occurred in this block, so that the difference of 69.4° F can be taken as representing the maximum temperature rise in any part of the concrete in which temperatures were measured in this dam. If thicker and heavier blocks of concrete had been placed at the same time, it is possible that the temperature rise might have been even greater. Variations in reservoir level during the entire period of record are shown in Table 11.

All the cores removed from the dam were taken from concrete which cooled naturally, that is, without artificial cooling. However, three cores subject to artificial coling were taken from the small test block mentioned in Item 1 under the heading, "General Observations." This block was 5 ft square by 4 ft high and was built with a 2-in. cored hole vertically through its center. It was provided with six thermometers for recording its temperature during setting and curing. After placing the concrete in the block, an insulated roof was put on it, and when the forms were removed after two days of setting, the sides of the block were also carefully protected with insulated housing. Inside the housing four 500-watt space heaters were placed to keep the temperature as nearly 126° F (the estimated temperature in the interior of a large block) as possible. A considerable number of pans and pails of water were also placed in the space around the block to keep the humidity high. The relative humidity during the test varied from 52 to 80% and after one month of curing the concrete was found to be thoroughly saturated; therefore, it did not dry out due to the heating. The circulation of cooling water through the block was begun as soon as the forms were stripped from it and continued without interruption throughout the 30-day test.

One cylinder made from the same concrete placed in the block was kept under laboratory conditions and tested at the end of 28 days, when it had a compressive strength of 2 475 lb per sq in. Three cylinders, which were made from the same concrete and cured one day in the moulds, two days in water at 70° F, and 30 days inside the housing with the test block, at approximately 126° F, averaged 3 180 lb per sq in. Three test cylinders, cast in stove-pipe sections placed vertically in the test block itself, were recovered at the end of the test and averaged 2 642 lb per sq in. Due to the special conditions under which all these test cylinders were made and cured, the results cannot be considered conclusive as to the relative streugth of concrete from cooled and non-cooled blocks. It is evident, however, that the cores taken from the small cooled block itself did have reasonably high compressive strength.

The writers agree with Messrs. Karpov and Templin that the determination of the actual stresses in the arch is the end to be sought, wherever possible. Neither time nor money was available to do this at Ariel Dam, so that a more moderate program was adopted. It is quite probable that the future will see stress tests carried out on structures of considerable size. The theory of arch-dam design will also continue to be changed and improved and will gradually come to cover some or all of the points mentioned by Messrs. Karpov and Templin, thus bringing theory and actuality closer together.

The writers wish to thank all those who have taken the time to discuss the paper and to contribute new data or ideas.

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WIND STRESSES BY SLOPE DEFLECTION AND CONVERGING APPROXIMATIONS

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WITH DISCUSSION BY MESSRS. G. A. MANEY, L. E. GRINTER, S. S. GORMAN, A. FLORIS, AND JOHN E. GOLDBERG.

SYNOPSIS

The purpose of this paper is to present a method of wind-stress analysis for rigid frames, based on the well-known slope-deflection analysis. By adjustments in the formulas and manipulation of the computations, rapid successive solutions are effected which after a few computations approach final accurate results, consistent with previously accepted refined but arduous processes.

The method is to be considered as basic or fundamental, directly applicable both to the analysis of stresses in individual bents acted upon by known horizontal forces and to structures having symmetrical layouts. For the latter type of problem a conception of a "composite bent" is introduced as a tool for structural analysis. Finally, a solution is proposed for secondary wind stresses, caused by column strain. The proposed methods have the merits of accuracy, speed, and practicability.

INTRODUCTION

Since its original presentation the slope-deflection method of wind-stress analysis by G. A. Maney, M. Am. Soc. C. E.,[2] has been considered to be one of the best of the so-called "exact" methods. As first presented, however, it was too laborious for general use. Many variations of the original have since been proposed, designed to eliminate the tedious work necessary for a solution of the great number of simultaneous equations which form the basis of the analysis and at the same time to retain the theoretical accuracy.

NOTE.—Published in May, 1933, *Proceedings*.

[1] Chicago, Ill.

[2] *Engineering Studies No. 1*, Univ. of Minnesota, 1915.

In addition, many admirable methods based on other theories have been proposed from time to time. Broadly speaking, however, because of elements of inaccuracy, impracticability, or lack of general application, no single method has enjoyed a widespread every-day use. The slope-deflection method suffered, with other methods, because of its tediousness, but it has always been used as a standard by which the accuracy of other and newer methods may be measured.

A more or less recent consideration which has added to the complications of an already involved problem is the effect of column strains under wind load. This paper is believed to present a simple, and yet theoretically correct, method of analysis, based on the fundamental slope-deflection theory, which can be made to include, when desired, the effect of column strains.

General Scope and Considerations

The treatment covers the following cases: (1) Direct analysis is applied to an individual bent; (2) when the structural layout of the whole building is symmetrical or approximately so, and particularly when the tower is an integral part of the structure, that is, not carried on trusses, the structure as a whole may be analyzed by means of the conception of the "composite bent" as explained later; and (3) when the tower is supported above the main part of the building by trusses, some assumption regarding the degree of fixity of the lower connections of the columns at the bottom of the tower having been made, it is possible to analyze the tower as a separate structure in which the trusses are analogous to foundations.

Whether the symmetrical or asymmetrical layout is the more important is, of course, a matter of opinion and personal experience; but it appears that a large percentage of the more important skyscrapers are either symmetrical in layout or approach symmetry and may be analyzed on that basis. In view of the fact, however, that apparently no simple and yet rational method of analyzing an extremely asymmetrical building as a unit is available, it would seem best, in problems of this type, to divide the design load between the various braced bents on some arbitrary or empirical basis.

Notation

For convenience, the various symbols are here collected and defined. Clockwise moments and angles may be assumed to be either positive or negative, as long as the same convention is observed throughout any analysis. The use of consistent units is presupposed. In interpreting the definitions, the reader should bear in mind the conception of the composite bent, as described subsequently. Following is the notation:

f = the ratio, for any column, of its relative stiffness, K_C, to ΣK_C for the story;

h = story height = length of column;

m = end moment: m_{AB} = actual moment at the end, or Joint A of a member, AB; that is, for example, m_{nX-mX} = actual moment at the end, or Joint nX of the member, $nX - mX$; m_{F-AB} = fixed-end moment at the end, or Joint A of the member, AB;

C = the ratio, for any member, of K to $\frac{2EI}{L}$, a constant for any analysis; it also equals the ratio of any rotation, in radians, to the same rotation in anomalous units;
D = relative transverse displacement of ends of member (horizontal for columns, vertical for beams and girders);
E = modulus of elasticity of the material of the members;
I = moment of inertia of the cross-section of the member;
K = relative stiffness of a member: K_{AB}, of the member, AB; K_i, of a member in which the rotation at the far end is θ_i; K_C, of a column; and K_G, of a girder (or beam):
L = length of a member (particularly of beams);
M = "wind moment" = horizontal shear on a story times the story height;
$R = \frac{D}{h}\left(\text{or } \frac{D}{L}\right)$ in anomalous units: R' = an initial or estimated value of R for any story;
S = rotation of a joint, in radians: S_A, of the joint, or end, A; and, S_B, of the joint, or end, B;
θ = rotation of a joint in anomalous units: θ_A, of Joint A; θ_B, of Joint B; θ', an initial, or estimated value of the average of all θ-values of any story; and θ_i, the rotation of a joint at the opposite end of a member that frames into the joint under consideration.

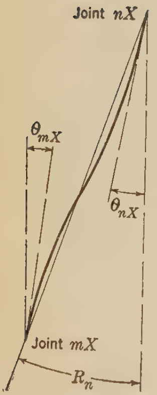


FIG. 1.—COLUMN DISTORTION.

A "story" consists of the columns, together with the girders or beams immediately above. For properties or variables of the various stories, columns, joints, or beams, subscripts are used. For example, the number of the story, counted up from the foundation, is designated by a lower case letter, such as m, n, or o, in formulas and equations, or by a number, in the actual analysis. Columns, joints, and beams or girders are similarly designated by an upper case letter, for example, A, B, C, ..., W, X, Y; for convenience, A is taken at the left. Thus, the Xth column of the nth story is designated as the nXth column; the third joint of the fourth story, as Joint $4C$; and the relative stiffness of the nXth column, as K_{CnX}. The relations of the elastic line of a distorted column are shown in Fig. 1.

FUNDAMENTAL EQUATIONS

For simplicity, the following theory is developed on the basis of the single bent, either individual or composite. The general equation on which the slope-deflection method of analysis is based is the expression for the moment at the end, A, of the member, AB:

$$m_{AB} = m_{F\text{-}AB} + \frac{2EI}{L}\left(3\frac{D}{L} - 2S_A - S_B\right) \quad \ldots\ldots\ldots\ldots(1)$$

in which, $\frac{2EI}{L}$ is the stiffness factor. In this form it includes the effects of: (1) Beam loading; (2) joint translation; and (3) joint rotation.

In the usual conception and handling of wind-stress problems, the designer is not generally concerned with beam loading; he dissociates vertical (dead and live) load and wind load, and also assumes that all loads are applied at the panel points. Thus, he eliminates the term, $m_{F\text{-}AB}$, from this particular application of the fundamental equation.

As a means of reducing the work to simpler figures, there is introduced into the formula the idea of relative stiffness, substituting K for $\frac{2EI}{L}$, and θ and R (which are in anomalous units) for the corresponding S and $\frac{D}{L}$ (which are in radians). Then, for the Xth column of the nth story:

$$m_{nX-mX} = K_{CnX}\,(3R_{nX} - 2\,\theta_{nX} - \theta_{mX})\ldots\ldots\ldots\ldots\ldots(2)$$

and for the girder (or beam) which lies between the joint, nX, and the joint, nY, neglecting the effect of column strains:

$$m_{nX-nY} = -\,K_{GnX}\,(2\,\theta_{nX} + \theta_{nY})\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3)$$

Where column strains, of known or assumed amount, are taken into account as causing relative transverse displacement of the ends or joints, the most convenient form is:

$$m_{nX-nY} = \frac{6EI}{L}\,\frac{D}{L} - K_{GnX}\,(2\,\theta_{nX} + \theta_{nY})\ldots\ldots\ldots\ldots(4)$$

By comparing Equations (1) and (2), it may be seen that the relation between any horizontal deflection (occurring in one column length) and its corresponding, R, is:

$$D = Rh\,\frac{K}{\dfrac{2EI}{h}} = CRh\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

Equation (5) is of use in calculating the actual deflections of the various stories of a building after the various R-values have been obtained.

THE BENT EQUATION

Because the ratio by which the total wind load is divided between the frame, and the walls, floors, etc., is an unknown and not easily estimated quantity, it is usual to assume that the frame carries the entire load, giving, in effect, an increased factor of safety or factor of reduction for both stresses and deflections.

Consider a single bent as being acted upon by known horizontal forces, and cut all the columns of the nth story by a horizontal plane. Then, applying the law of statics, $\Sigma H = 0$, to the part of the bent above the cut section,

$$\Sigma\ (\text{column shears}) = \text{external shear}$$

in which, the external shear is the total of all loads above the cutting plane. Multiplying both sides of the equation by h, the story height,

$$\Sigma\ (\text{column shears})\ (h) = (\text{external shear})\ (h) = M_n$$

Since shear is the rate of change of moment,

$$\Sigma\ (\text{both end moments of all columns}) = M_n$$

By Equation (2), the sum of both end moments of the *nXth* column is,

$$K_{CnX}\,(6\,R_{nx} - 3\,\theta_{nx} - 3\,\theta_{mx})$$

and for all columns of the *n*th story, since R_n is the same for all when they are of a uniform height,

$$6\,\Sigma\,K_{Cn}\,R_n - \Sigma\,[K_{CnX}\,(3\,\theta_{nX} + 3\,\theta_{mX})] = M_n \,\ldots\ldots\ldots\ldots (6)$$

This becomes the standard form of the bent equation.

The Joint Equation

To each joint the law of statics, $\Sigma\, m = 0$, may be applied. Take the joint, *nX*, as typical. Then,

$$m_{nx\text{-}nw} = -\,K_{Gnw}\,(2\,\theta_{nx} + \theta_{nw})$$

$$m_{nx\text{-}ny} = -\,K_{Gnx}\,(2\,\theta_{nx} + \theta_{ny})$$

$$m_{nx\text{-}mx} = K_{Cnx}\,(3\,R_n - 2\,\theta_{nx} - \theta_{mx})$$

$$m_{nx\text{-}ox} = K_{Cox}\,(3\,R_o - 2\,\theta_{nx} - \theta_{ox})$$

Hence, by addition of these four equations,

$$\Sigma\, m = 3\,K_{Cnx}\,R_n + 3\,K_{Cox}\,R_o - 2\,\Sigma\,K\,\theta_{nx} - \Sigma\,(K_i\,\theta_i) = 0 \ldots (7)$$

in which, $\Sigma\,(K_i\,\theta_i)$ is the summation of the products of the *K*-values for each member framing into the joint under consideration times the θ-values for the joints at the respective far ends of those members. Equation (7) is the general form of joint equation as used in primary wind-stress analysis.

Methods of Solution

The original method of solution consisted of writing a bent equation for each story and a joint equation for each joint, solving these equations simultaneously for the various *R* and θ-values. The *R* and θ-values thus obtained were substituted in expressions which had the form of Equations (2) and (3), to determine the correct end moments throughout the bent. Obviously, for any but the simplest bents, the labor involved in such a solution would be prohibitive.

It is well known that, under wind load, the points of contraflexure, particularly of columns, tend to approach the mid-lengths of members. The method to be outlined herein takes due account of this action; but, by using it only to obtain initial values of the various *R*'s and initial average values of the θ's of each story, it does not, so to speak, place too much confidence in this fact.

Taking the part of the bent adjacent to the *m*th story, and assuming all joints to have the same rotation, θ'_m, the bent equations for the *m*th and *n*th stories become,

$$M_m = \Sigma\,K_{Cm}\,(6\,R_m - 6\,\theta'_m)$$

$$M_n = \Sigma\,K_{Cn}\,(6\,R_n - 6\,\theta'_m)$$

or, by addition,

$$M_m + M_n = 6\,\Sigma K_{Cm}\,R_m + 6\,\Sigma K_{Cn}\,R_n - 6\,\theta'_m\,(\Sigma\,K_{Cm} + \Sigma K_{Cn})\ldots(8)$$

The joint equation for any joint of the mth story becomes,

$$3\,K_{Cm}\,R_m + 3\,K_{Cn}\,R_n - 3\,\Sigma\,K\,\theta'_m = 0$$

and for all joints of the mth story, taken together:

$$3\,\Sigma\,K_{Cm}\,R_m + 3\,\Sigma\,K_{Cn}\,R_n - \Sigma\,(3\,\Sigma\,K)\theta'_m = 0\ldots\ldots\ldots\ldots(9)$$

Eliminating R_m and R_n between Equations (8) and (9):

$$\Sigma\,(6\,\Sigma\,K)\theta'_m = M_m + M_n + 6\,\theta'_m\,(\Sigma\,K_{Cm} + \Sigma\,K_{Cn})$$

Since $\Sigma\,(6\,\Sigma K) = 12\,\Sigma K_{Gm} + 6\,\Sigma K_{Cm} + 6\,\Sigma K_{Cn}$, this equation reduces to:

$$\theta'_m = \frac{M_m + M_n}{12\,\Sigma K_{Gm}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(10)$$

This gives a formula for the initial average values of the joint rotations in terms of known quantities.

It is customary to assume a degree of fixity for the columns at the foundations, generally either pin-ended or fully fixed, and for these conditions the foregoing derivation may be modified to obtain an expression for θ'_1. For the pin-ended condition, Equation (10) becomes:

$$\theta'_1 = \frac{2\,M_1 + M_2}{12\,\Sigma K_{G1}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(10a)$$

and for the fully fixed condition:

$$\theta'_1 = \frac{M_1 + M_2}{12\,\Sigma K_{G1} + \Sigma K_{C1}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots(10b)$$

Proceeding to a simplified conception of the bent equation, the standard Equation (6) for R_n is solved to obtain:

$$R_n = \frac{M_n}{6\,\Sigma K_{Cn}} + \frac{\Sigma\,[K_{CnX}\,(\theta_{mX} + \theta_{mX})]}{2\,\Sigma K_{Cn}}\ldots\ldots\ldots\ldots(11)$$

which may be written in the form:

$$R_n = \frac{M_n}{6\,\Sigma K_{Cn}} + (\text{weighted average value of all } \theta_m\text{'s and } \theta_n\text{'s})\ldots\ldots(11a)$$

For calculating an R-value from any series of θ-values in an actual analysis, the following is perhaps the best form:

$$R_n = \frac{M_n}{6\,\Sigma K_{Cn}} + \Sigma\left[f_{nX}\,\frac{(\theta_{mX} + \theta_{nX})}{2}\right]\ldots\ldots\ldots\ldots(11b)$$

in which, f_{nX} is the ratio, for the Xth column of the nth story, of its K-value to $\Sigma\,K_{Cn}$. For the initial value of R_n, in terms of initial θ-values:

$$R'_n = \frac{M_n}{6\,\Sigma K_{Cn}} + \frac{\theta'_m + \theta'_n}{2}\ldots\ldots\ldots\ldots\ldots\ldots(11c)$$

The most convenient form of the joint equation is obtained by solving Equation (7) for its corresponding θ:

$$\theta_{nx} = \frac{3\,K_{Cnx}\,R_n + 3\,K_{Cox}\,R_o - \Sigma\,(K_t\,\theta_t)}{2\,\Sigma\,K} \quad \ldots\ldots\ldots\ldots(12)$$

Equations (10) (and (10*a*) or (10*b*)), and (11*c*), therefore, are of use in estimating the θ and R-values of each story. Equations (12) and (11), (11*a*), or (11*b*) are used to determine more nearly correct θ and R-values, and, if the work is carried far enough, absolutely correct values may be obtained from these equations.

ORDER OF PROCEDURE

The recommended procedure in a primary wind-stress analysis may now be outlined. Each consecutive step is numbered for convenience. The same numbering is applied to the illustrative problem shown in Fig. 2 and explained subsequently. The procedure is as follows:

(1) Calculate the relative stiffness of each member of the frame; also, calculate $\Sigma\,K_G$ and $\Sigma\,K_C$ for each story, $\Sigma\,K$ for each joint, and f for each column.

(2) Determine the horizontal shear on each story, and from this determine $M =$ shear times story height for each story.

(3) Calculate θ' for each story, using Equation (10) for all stories above the first and Equation (10*a*) or Equation (10*b*) for the first story.

(4) Calculate R' for each story, using Equation (11*c*).

(5) Calculate $3\,K_C\,R'$ for each column. (For subsequent cycles use $3\,K_C\,R$.)

(6) Separate the joints into two groups of alternate joints—one group to include Joints 1*A*, 1*C*, 1*E*, etc., 2*B*, 2*D*, etc., and the other group to include the remaining joints. For each joint of one group calculate the quantity, $\frac{\Sigma\,(3\,K_C R')}{3\,\Sigma\,K}$, which is the form Equation (12) takes when the points of contraflexure are at the mid-lengths of the members. This is to be considered in the nature of an estimate. For the joints of the first story, instead of the foregoing expression use:

$$\frac{3\,K_{C2}\,R'_2 + \frac{3}{2}\,K_{C1}\,R'_1}{3\,\Sigma K - \frac{3}{2}\,K_{C1}}$$

for columns pin-ended at the foundations; and,

$$\frac{\Sigma\,(3\,K_C\,R')}{3\,\Sigma K - K_{C1}}$$

for columns fully fixed at the foundations. These expressions, easily derived from fundamental slope-deflection considerations, more nearly approximate the true θ-value of a joint under the assumed conditions of fixity.

(7) Using Equation (12), calculate a series of θ-values for the remaining joints, based on the results of Steps (5) and (6).

(8) By the same method as Step (7), calculate a series of θ-values for the first group of joints, based on the results of Steps (5) and (7).

(9) For greater accuracy, the θ-values of Step (7) may be recalculated on the basis of Step (8). Ordinarily, even in the analysis of asymmetrical bents, this step may be omitted as it was in the illustrative problem of Fig. 2.

(10) Using the results of Steps (7) and (8), or of Steps (8) and (9), in Equations (11), (11*a*), or (11*b*), calculate a new set of R-values (in reality this is the first approximation for these R-values, the corresponding R'-values being considered as estimates).

(11) Repeat Steps (5), (7), and (8) (and Step (9) if extreme accuracy is desired) on the basis of Step (10), using for each θ_i in Equation (12) the last value obtained. This gives a new series of θ-values.

(12) Repeat Step (10), using in Equations (11), (11*a*), or (11*b*) the results of Step (11). This gives a new series of R-values.

(13) Repeat Steps (11) and (12) until the desired degree of accuracy is obtained. It will be found that for all but the most irregular cases this step may be omitted. In fact, in many analyses, the desired accuracy will be obtained if the approximations or cycles proceed no further than Step (10).

(14) Substituting the results of the previous steps in Equations (2) and (3), calculate the end moments of each member.

(15) Calculate, if desired, the horizontal deflection of each story, using Equation (5), and working upward from the first story.

Illustrative Problem

It is desired to find the end moments of the members in the lower part of a 20-story bent, as shown in Fig. 2. The wind load, which causes the panel concentrations and shears as shown, is equivalent to a pressure of 30 lb per sq ft on a strip 1 ft wide and extending to the top of the building. All work is to be done on a simple line drawing of the bent (this will be found to be the most convenient method). The columns are to be assumed as fully fixed at the foundations. In order to provide a limit to the problem, it will be assumed that the points of contraflexure of the fourth-story columns are at their mid-lengths; also, it will be assumed that $R_4 = R_3$.

The fourth story is used to show the general form and detail of the calculations. The encircled figures refer to the steps of the foregoing recommended procedure. When considered with this step-by-step procedure, Fig. 2 becomes practically self-explanatory.

Deflection

The deflection at the top of any story is, of course, ΣD up to and including that story. Equation (5) gives the value of any D in terms of its corresponding R and h; that is, $D = CRh$, in which, C is the ratio of the relative stiffness factor to the actual stiffness factor $\left(\text{which is taken as } \frac{2EI}{L}\right)$. Thus, in the problem of Fig. 2, the value of $\frac{I}{L}$ was taken as the K-value for

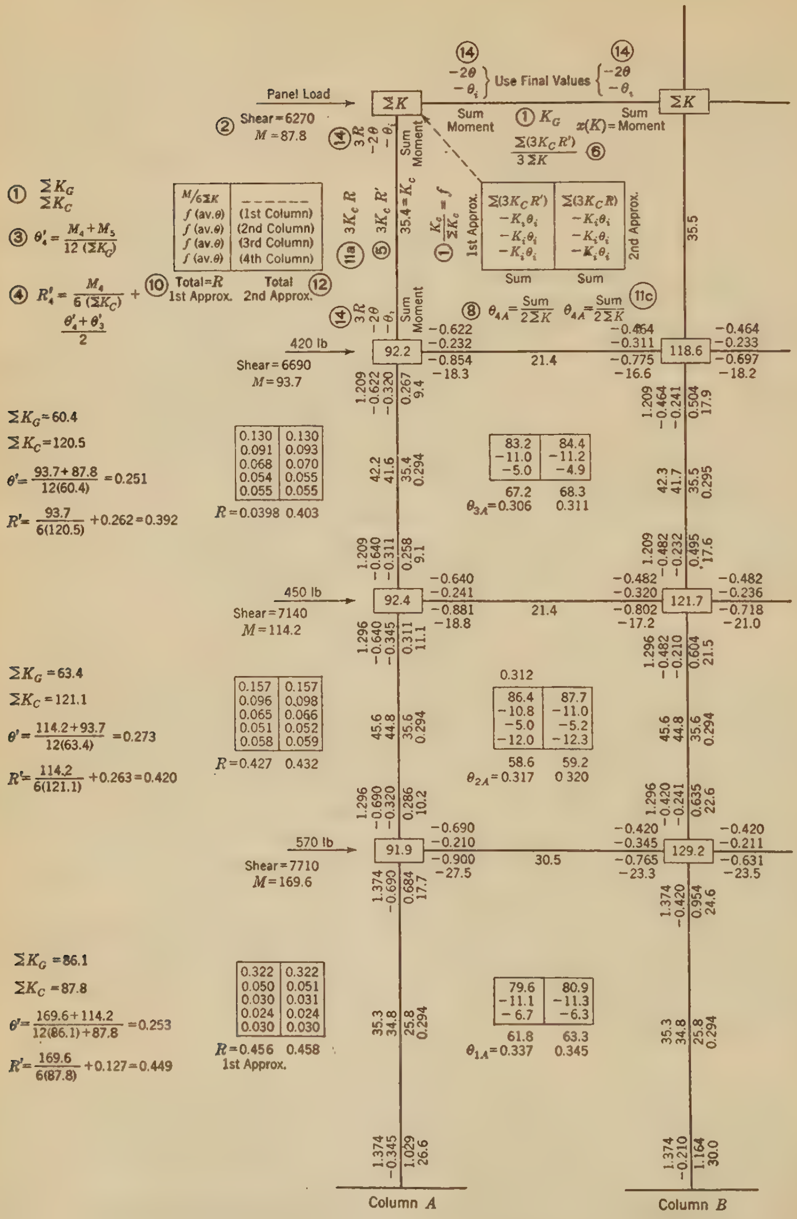
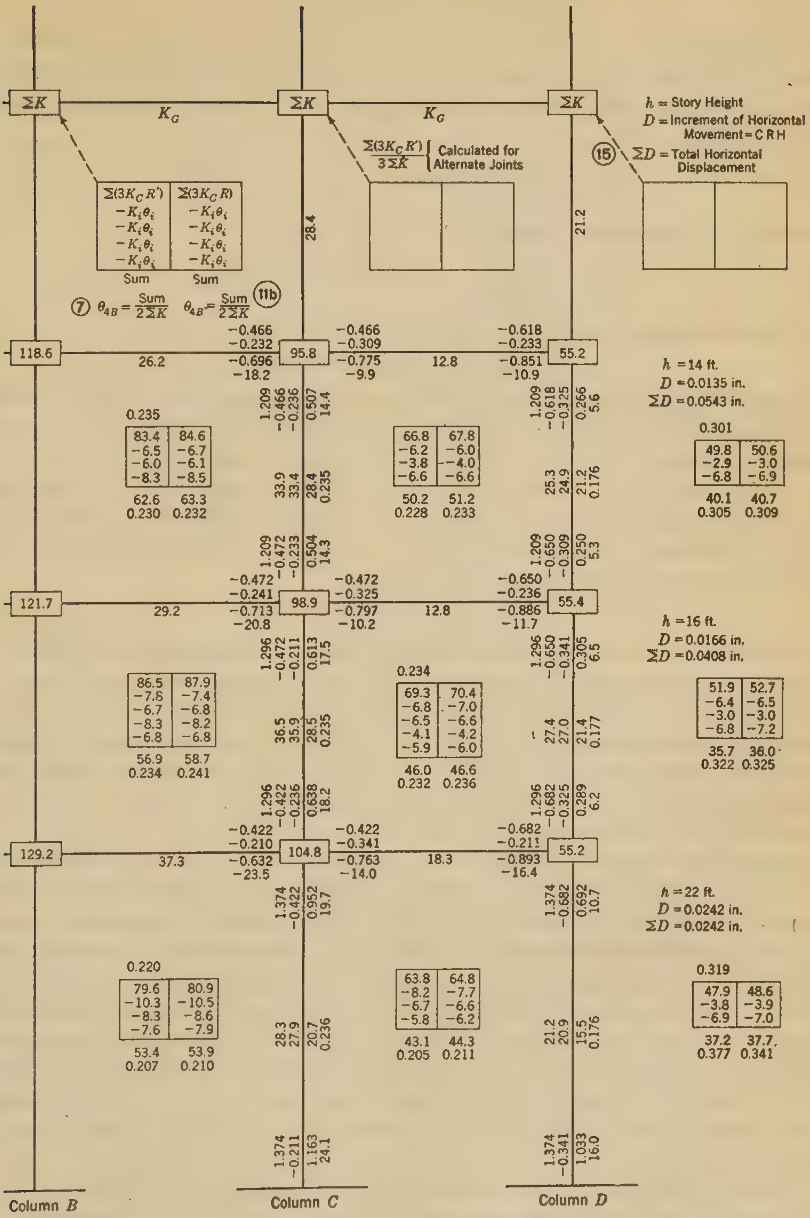

⑭ −2θ −θᵢ } Use Final Values { −2θ −θᵢ ⑭
Panel Load
ΣK
ΣK
② Shear=6270 M=87.8
⑭ 3R −2θ −θᵢ
Sum Moment
Sum Moment
① K_G
Sum x(K)=Moment
Σ(3K_C R′) / 3ΣK ⑥
① ΣK_G / ΣK_C
③ θ′₄ = (M₄+M₅) / 12(ΣK_G)
④ R′₄ = M₄ / 6(ΣK_C) + (θ′₄+θ′₃)/2
M/6ΣK
f (av.θ) (1st Column)
f (av.θ) (2nd Column)
f (av.θ) (3rd Column)
f (av.θ) (4th Column)
⑩ Total=R 1st Approx.
Total ⑫ 2nd Approx.
⑪a 3K_c R
⑤ 3K_c R′
35.4=K_c
① K_c/ΣK_c = f
1st Approx.
Σ(3K_C R′) −Kᵢθᵢ −Kᵢθᵢ −Kᵢθᵢ Sum
Σ(3K_C R) −Kᵢθᵢ −Kᵢθᵢ −Kᵢθᵢ Sum
2nd Approx.
35.5
⑭ 3R −2θ −θᵢ
Sum Moment
⑧ θ₄A = Sum/2ΣK
θ₄A = Sum/2ΣK ⑪c
420 lb
Shear=6690 M=93.7
92.2
−0.622 −0.232 −0.854 −18.3
21.4
−0.464 −0.311 −0.775 −16.6
118.6
−0.464 −0.233 −0.697 −18.2
1.209 −0.622 −0.320
0.267 9.4
1.209 −0.464 −0.241
0.504 17.9
ΣK_G=60.4
ΣK_C=120.5
θ′= (93.7+87.8)/12(60.4) =0.251
R′= 93.7/6(120.5) +0.262=0.392
0.130 0.130
0.091 0.093
0.068 0.070
0.054 0.055
0.055 0.055
R=0.0398 0.403
42.2 41.6
35.4 0.294
83.2 84.4
−11.0 −11.2
−5.0 −4.9
67.2 68.3
θ₃A=0.306 0.311
42.3 41.7
35.5 0.295
1.209 −0.640 −0.311
0.258 9.1
1.209 −0.482 −0.232
0.495 17.6
450 lb
Shear=7140 M=114.2
92.4
−0.640 −0.241 −0.881 −18.8
21.4
−0.482 −0.320 −0.802 −17.2
121.7
−0.482 −0.236 −0.718 −21.0
1.296 −0.640 −0.345
0.311 11.1
1.296 −0.482 −0.210
0.604 21.5
ΣK_G =63.4
ΣK_C =121.1
θ′= (114.2+93.7)/12(63.4) =0.273
R′= 114.2/6(121.1) +0.263=0.420
0.157 0.157
0.096 0.098
0.065 0.066
0.051 0.052
0.058 0.059
R=0.427 0.432
45.6 44.8
35.6 0.294
0.312
86.4 87.7
−10.8 −11.0
−5.0 −5.2
−12.0 −12.3
58.6 59.2
θ₂A=0.317 0 320
45.6 44.8
35.6 0.294
1.296 −0.690 −0.320
0.286 10.2
1.296 −0.420 −0.241
0.635 22.6
570 lb
Shear=7710 M=169.6
91.9
−0.690 −0.210 −0.900 −27.5
30.5
−0.420 −0.345 −0.765 −23.3
129.2
−0.420 −0.211 −0.631 −23.5
1.374 −0.690
0.684 17.7
1.374 −0.420
0.954 24.6
ΣK_G =86.1
ΣK_C =87.8
θ′= (169.6+114.2)/12(86.1)+87.8 =0.253
R′= 169.6/6(87.8) +0.127=0.449
0.322 0.322
0.050 0.051
0.030 0.031
0.024 0.024
0.030 0.030
R=0.456 0.458
1st Approx.
35.3 34.8
25.8 0.294
79.6 80.9
−11.1 −11.3
−6.7 −6.3
61.8 63.3
θ₁A=0.337 0.345
35.3 34.8
25.8 0.294
1.374 −0.345
1.029 26.6
1.374 −0.210
1.164 30.0
Column A
Column B

FIG. 2.—ILLUSTRATIVE PROBLEM, LOWER PART OF TWENTY-STORY, ASYMMETRICAL BENT, GENERAL FORM AND STEP-BY-STEP


ΣK
ΣK
ΣK
K_G
K_G
Σ(3K_CR')
−$K_i\theta_i$
−$K_i\theta_i$
−$K_i\theta_i$
−$K_i\theta_i$
Sum
Σ(3K_CR)
−$K_i\theta_i$
−$K_i\theta_i$
−$K_i\theta_i$
−$K_i\theta_i$
Sum
⑦ $\theta_{4B} = \frac{\text{Sum}}{2\Sigma K}$
$\theta_{4B} = \frac{\text{Sum}}{2\Sigma K}$ (11b)
$\frac{\Sigma(3K_C R')}{3\Sigma K}$ Calculated for Alternate Joints
(15)
h = Story Height
D = Increment of Horizontal Movement = C R H
ΣD = Total Horizontal Displacement
28.4
21.2
118.6
26.2
−0.466
−0.232
−0.696
−18.2
95.8
−0.466
−0.309
−0.775
−9.9
12.8
−0.618
−0.233
−0.851
−10.9
55.2
h = 14 ft.
D = 0.0135 in.
ΣD = 0.0543 in.
1.209
−0.466
−0.236
0.507
14.4
1.209
−0.618
−0.325
0.266
5.6
0.235
83.4
−6.5
−6.0
−8.3
84.6
−6.7
−6.1
−8.5
62.6
0.230
63.3
0.232
33.9
33.4
28.4
0.235
66.8
−6.2
−3.8
−6.6
67.8
−6.0
−4.0
−6.6
50.2
0.228
51.2
0.233
25.3
24.9
21.2
0.176
0.301
49.8
−2.9
−6.8
50.6
−3.0
−6.9
40.1
0.305
40.7
0.309
1.209
−0.472
−0.233
0.504
14.3
1.209
−0.650
−0.309
0.250
5.3
121.7
29.2
−0.472
−0.241
−0.713
−20.8
98.9
−0.472
−0.325
−0.797
−10.2
12.8
−0.650
−0.236
−0.886
−11.7
55.4
h = 16 ft.
D = 0.0166 in.
ΣD = 0.0408 in.
1.296
−0.472
−0.211
0.613
17.5
1.296
−0.650
−0.341
0.305
6.5
86.5
−7.6
−6.7
−8.3
−6.8
87.9
−7.4
−6.8
−8.2
−6.8
56.9
0.234
58.7
0.241
36.5
35.9
28.5
0.235
0.234
69.3
−6.8
−6.5
−4.1
−5.9
70.4
−7.0
−6.6
−4.2
−6.0
46.0
0.232
46.6
0.236
27.4
27.0
21.4
0.177
51.9
−6.4
−3.0
−6.8
52.7
−6.5
−3.0
−7.2
35.7
0.322
36.0
0.325
1.296
−0.422
−0.236
0.638
18.2
1.296
−0.682
−0.325
0.289
6.2
129.2
37.3
−0.422
−0.210
−0.632
−23.5
104.8
−0.422
−0.341
−0.763
−14.0
18.3
−0.682
−0.211
−0.893
−16.4
55.2
h = 22 ft.
D = 0.0242 in.
ΣD = 0.0242 in.
1.374
−0.422
0.952
19.7
1.374
−0.682
0.692
10.7
0.220
79.6
−10.3
−8.3
−7.6
80.9
−10.5
−8.6
−7.9
53.4
0.207
53.9
0.210
28.3
27.9
20.7
0.236
63.8
−8.2
−6.7
−5.8
64.8
−7.7
−6.6
−6.2
43.1
0.205
44.3
0.211
21.2
20.9
15.5
0.176
0.319
47.9
−3.8
−6.9
48.6
−3.9
−7.0
37.2
0.377
37.7
0.341
1.374
−0.211
1.163
24.1
1.374
−0.341
1.033
16.0
Column B
Column C
Column D

SHOWING COMPLETE PRIMARY WIND-STRESS CALCULATIONS. FOURTH STORY INDICATES PROCEDURE OF COMPUTATIONS.

each member, and:

$$C = \frac{1.}{2\,E} = \frac{1}{60\,000\,000}$$

Furthermore, if, as in the illustrative problem, the deflections are desired in inches while the moments are in units of 1 000 lb-ft and the story height is in feet, C must be multiplied by 144 000. The deflection at the top of the first story would be:

$$D = \frac{144\,000}{60\,000\,000}\,(0.458)\,(22) = 0.0242 \text{ in.}$$

Similarly, for the second story,

$$D = \frac{144\,000}{60\,000\,000}\,(0.432)\,(16) = 0.0166 \text{ in.}$$

The total horizontal deflection at the top of the second story is:

$$\Sigma\,D = 0.0242 + 0.0166 = 0.0408 \text{ in.}$$

Since the design load was the wind pressure on a vertical strip, 1 ft wide, extending to the top of the building, true deflections will be given by the product of these values and the distance, in feet, between bents.

Often, perhaps as a preliminary check on the rigidity of the structure, the engineer may desire the value of the primary deflection in advance of having calculated the actual R-values. For this purpose the R'-values, which are easily calculated, may be used.

THE COMPOSITE BENT

Thus far, a bent has been considered as being the structural unit. In reality, the action of any individual bent may depend quite as much on the action of neighboring bents as it does on the loads directly imposed. In tall buildings, particularly, the floors and walls force all bents to assume approximately the same horizontal deflections. The division of load between the various bents, then, should be made on the basis of some rational conception of unified action throughout the frame.

One of the simplest, yet most rational, methods of analyzing this unified action of a complete frame involves the conception of a composite bent. The general method, as applied to buildings of symmetrical structural layout, includes, briefly, the following procedure: (*a*) Group all bents into one composite bent (the loads, of course, must also be grouped together); (*b*) obtain a fairly accurate series of R-values—in most cases, the initial R'-values will be adequate—for this composite bent; (*c*) using these R-values in the individual bents, calculate a set of consistent θ-values for the various joints; (*d*) calculate a new series of R-values on the basis of the weighted average of the columns of all bents; (*e*) repeat Steps (*c*) and (*d*), if desired; and, (*f*) compute moments.

In the case of special problems, the method is subject to variation. When all parallel bents are exactly the same, a single bent may be analyzed for its proportion of the total load. On the other hand, when the building is completely asymmetrical, some corrective factor may be introduced to take care of the torsion of the frame as a whole.

SECONDARY WIND STRESSES

As the frame deflects under the action of the wind load, all beams tend to have the same general effect on the columns; that is, uplift on the windward side and downward thrust on the leeward column. Because of differences in beam reactions, the net effect is a variation in column loads and, consequently, in column strains throughout each story. As a result, the various joints of any story are forced to different levels, somewhat as shown in Fig. 3, and certain "unbalanced" moments are induced in the beams. This condition becomes extremely important in tall buildings although it may be practically or even altogether negligible in lesser structures.

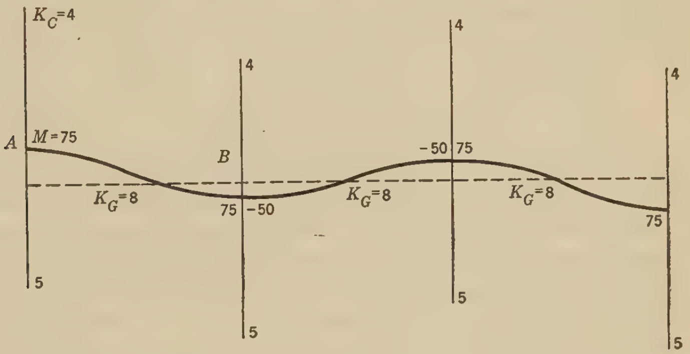


FIG. 3.—BEAM DISTORTION CAUSED BY COLUMN STRAIN.

For any rational anaylsis of these secondary stresses it is necessary to know the actual column loads; that is, the primary loads minus the secondary loads. One simple plan, however, is to use the column strains induced by the primary beam moments, unrelieved by the secondary beam moments, as a basis of solution, this, of course, being the most severe case and the absolute maximum limit of secondary stresses. Obviously, this method may lead, in some cases, to excessive values of secondary stresses. This is particularly true of the taller and more slender buildings. Perhaps the most logical procedure for a secondary stress analysis which is to take account of the relieving effects throughout the bent or frame is somewhat as follows: (1) Make a rough secondary stress solution on the basis of unrelieved primary column strains; (2) correct these primary column strains for the relieving effect of this rough secondary solution; and, (3) use this "relieved" condition as a basis for an accurate secondary stress analysis.

The final result in any case should be, as indicated in Step (3), an accurate analysis of secondary stresses, the approximate or rough solutions of the previous steps being used merely to obtain a more nearly correct basis for the final solution. In those analyses in which the unrelieved primary strains are used, Steps (1) and (2) are omitted. On the other hand, for important cases, Steps (1) and (2) may be repeated as often as necessary in order to obtain the desired accuracy.

The secondary stress analysis, whether it be a rough preliminary solution or an accurate final solution, must consider the following: (*a*) There exists at each end of each beam an unbalanced fixed-end moment equal to $\frac{6EI}{L}\frac{D}{L}$, in which, D is the relative vertical displacement of the ends of the beam; that is, the cumulative differential column strains up to that beam; (*b*) the joints, therefore, must rotate to new positions of equilibrium; and, (*c*) the various stories must deflect horizontally so that at each story the total shear in the columns, in so far as the secondary stresses are concerned, is zero.

As far as Point (*c*) is concerned, the solution is analogous to a wind-stress problem with no wind load. Then,

$$R_n = (\text{weighted average of } \theta_m\text{'s and } \theta_n\text{'s}) \ldots\ldots\ldots\ldots (13a)$$

or,

$$R_n = \Sigma\left[f_{nX}\,\frac{(\theta_{mX}+\theta_{nX})}{2}\right] \ldots\ldots\ldots\ldots (13b)$$

As an illustration of a simplified method of analysis, suppose that a certain story of a symmetrical three-span bent is so distorted by cumulative columns strains that the unbalanced moments indicated in Fig. 3 are induced. Bearing in mind the fact that the points of contraflexure are near the mid-lengths of the columns (justifying the use of $3\,K_C$ in the denominators), values of R, θ_A, and θ_B may be found by successive approximation. If R for the column is assumed to be zero (the beam R-values are, of course, known and of such amount as to give for the respective values of $3\,K_G R$, the quantities, 75 and —50), Equation (12) would give for a first approximation,

$$\theta_A = \frac{75 - \dfrac{8\,(25)}{2\,(8) + 3\,(4+5+8)}}{2\,(8) + 3\,(4+5)} = 1.68$$

The numerator will be recognized as consisting of $3\,K_G R$ (which is 75) minus K_i (which is 8) times an estimated value of θ_i (which is, in this case, θ_B, all other θ_i values being assumed to be equal to θ_A). The foregoing value of θ_A would give,

$$\theta_B = \frac{25 - 8\,(1.68)}{2\,(8) + 3\,(4+5+8)} = 0.17$$

Rechecking the value of θ_A, a step which is not generally necessary:

$$\theta_A = \frac{75 - 8\,(0.17)}{2\,(8) + 3\,(4+5)} = 1.71$$

By Equation (13*a*) or Equation (13*b*), these would give R approximately equal to 0.94, which, in turn, gives: $3\,K_{Cm}R = 3\,(0.94)\,5 = 14.1$, and, $3\,K_{Cn}R = 3\,(0.94)\,4 = 11.3$.

From the foregoing values the corrected joint rotations would be found from Equation (12), thus:

$$\theta_A = \frac{75 + 11.3 + 14.1 - \dfrac{8\,(25 + 14.1 + 11.3)}{2\,(8) + 3\,(4+5+8)}}{2\,(8) + 3\,(4+5)} = 2.22$$

and,

$$\theta_B = \frac{25 + 14.1 + 11.3 - 8\,(2.22)}{2\,(8) + 3\,(4 + 5 + 8)} = 0.34$$

Rechecking the value of θ_A:

$$\theta_A = \frac{75 + 14.1 + 11.3 - 8\,(0.34)}{2\,(8) + 3\,(4 + 5)} = 2.27$$

From these values a new R-value may be computed, giving, in this case, approximately 1.30.

From this point forward in the analysis, several variations of method may be used, depending on the accuracy desired:

(1) If the analysis is to be used merely as a preliminary estimate which will furnish a basis for calculating "relieved," or net, column strains, the values of R, θ_A, and θ_B, as found, may be used in Equations (2) and (4) to compute moments. It should be noted that, in such a preliminary solution, it is necessary to calculate only the beam moments.

(2) For the many cases in which the secondary stresses are relatively unimportant, although appreciable, the foregoing values may be used to calculate the moments.

(3) Where a more accurate solution is required, the foregoing method serves as a convenient means of obtaining a series of initial R-values, corresponding to the R'-values of the primary solution, on the basis of which one or more cycles may be applied to obtain final correct values of the secondary displacements. Moments may then be computed from these final values.

On the other hand, a series of cycles may be applied directly to the bent as a whole instead of splitting it up into sections, one of which has been analyzed previously. This straightforward method needs no explanation. Regardless of which method is used, Equation (13*a*) or Equation (13*b*) and a modification of Equation (12) which includes $3\,K_G R$, or $\frac{6\,EI}{L}\,\frac{D}{L}$, as well as $3\,K_C R$, will form the basis of solution.

CONCLUSION

The writer has presented, in the foregoing, a basic method for the analysis of both primary and secondary wind stresses. It must be borne in mind, however, that for some few special cases of the frame as a whole the method is subject to variation, as has been indicated.

This method, it is believed, offers certain definite advantages not generally possessed by other methods now in use, namely: (1) Speed and simplicity are obtained without the sacrifice of accuracy; (2) deflections are obtained directly; (3) the methods require no elaborate instruments; (4) with the conception of the composite bent, the method is easily extended to include the analysis of certain types of complete frames; and, (5) asymmetrical bents are analyzed quite as easily as symmetrical bents.

DISCUSSION

G. A. MANEY,[3] M. AM. SOC. C. E. (by letter).—A brief statement concerning the history of the use of converging approximations for rigid frame analyses might be interesting. Undoubtedly, this system dates about as far back as the problem of solving simultaneous equations.

Before March, 1915, when the writer first proposed his generalization of the slope-deflection method,[4] Mohr's method, with a rapidly converging approximate solution, was used by Allston Dana, M. Am. Soc. C. E., to obtain secondary stresses in the long-span Kenova Bridge.[5] Since that time, the writer and others have used various schemes for converging solutions.

The reasons for convergence are not subtle mathematical ones, but simply physical. The slope-deflection equation for a member loaded transversely and with end rotations, but no rotation of member as a whole, is:

$$M_{AB} = M_F/_{AB} - K\ (2\theta_A + \theta_B) \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which, the coefficient, K, varies as $\frac{I}{L}$ and θ is multiple of angle.

This equation written in long hand states that the final bending moment, M_{AB}, at the end, A, of any member, AB, is equal first to the value, $M_F/_{AB}$, which is the moment occurring at the end, A, due to intermediate transverse loads when neither Joint A nor Joint B are allowed to rotate. Then, as Joint A is allowed to rotate through the angle, θ_A, a moment equal to $(K)\ (2\ \theta_A)$ is superimposed on top of $M_F/_{AB}$ at the end, A. Incidentally, this angle, θ_A, induces only one-half as much moment at the end, B. Hence, when the end, B, is rotated through the angle, θ_B, only $(K)\ (\theta_B)$ is induced at the end, A.

The minus sign is really not important, but as stated in Equation (14), means that if θ_A is a clockwise rotation of the joint, A, the induced moment tends to rotate the joint, A, counter-clockwise.

The real and only reasons for convergence are contained in the expression,

$$M_{AB} = -\ (K)\ (2\theta_A + \theta_B) \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which, M_{AB} is used as a correction or addition to an original or known starting value which is a function of the member rotation as a whole (with the zero member end rotations) for wind load and secondary stress problems, and which is M_F (the well known "fixed beam" moment) for ordinary building frames under gravity loads.

The reasons for convergence may be stated, as follows:

(1) In one extreme case, where the end, B, is free to rotate, an angular change, θ_A, applied at the end, A, results in only one-half as large an angular change, θ_B, at the end, B.

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[3] Prof., Structural Eng., Northwestern Univ., Evanston, Ill.

[4] *Studies in Engineering No. 1*, Univ. of Minnesota, 1915.

[5] "Statically Indeterminate Stresses," by J. I. Parcel and G. A. Maney, Members Am. Soc. C. E., N. Y., 1926, p. 330.

(2) In the other extreme case, for the end, *B*, where it is fully fixed, an applied bending moment, M_{AB}, at the end, *A*, results in only one-half as much bending moment, M_{BA}, at the end, *B*.

(3) When corrections in either angular change or bending moment are applied to either the original zero angular values or the original M_F-value, such correction (or error in such correction) is felt by only one-half the amount of the correction at joints once removed (opposite end); by only one-quarter at joints twice removed; and by only one-eighth at joints three times removed (three member lengths away).

Examining Equations (14) and (15) it will be seen that whether one goes from the starting point, $M_F/_{AB}$, to the final value, M_{AB}, with a series of well controlled "guesses" by means of correcting M_F directly or by means of correcting θ-values, no essential difference in method is involved. Advocates of the so-called "moment-distribution" method prefer correcting M directly and it is possible that this procedure will alleviate the strain on the imagination which might be necessitated to visualize the physical distortions of the structure.

The writer wishes to commend Mr. Goldberg on having produced what is, in his estimation, the most expeditious convergence scheme yet applied to a frame under wind loads.

The writer and Mr. Goldberg have made a comparison between the moment-distribution "method" and the angular correction method used in the present paper.[6] It will be clearly seen by this comparison that they merely typify two methods which can be used advantageously to travel from the "fixed beam" moment of the slope-deflection method to the final moment by a quick and self-checking series of converging approximations.

L. E. GRINTER,[7] ASSOC. M. AM. SOC. C. E. (by letter).—The author has presented a method of solving for the joint rotations and lateral deflections of a building frame acting under wind load by successive substitution in approximate and exact equations. The title of the paper will lead most readers to expect a closer relationship to the general slope-deflection method than really exists. The author does use the general slope-deflection equations in the derivation of his working formulas (Equations (10) and (11*c*)), but, as the writer has shown,[8] such working formulas are more readily obtained by area moments. However, since the unknowns to be determined are rotations and deflections rather than moments, the name, "slope deflection," is properly descriptive of the method.

Thirteen steps are listed in the paper before one arrives at a final set of joint rotations and lateral deflections. Actually, this outline represents more than thirteen operations since Step (11) involves the repetition of three previous steps and Step (13) involves the repetition of two. The author indicates that Step (13) is usually unnecessary and that Steps (11) and (12) may be omitted for regular frames. Obviously, a criterion is found in Equation (11*c*). If this equation accurately predicts the lateral deflections of the frame (which will be true for frames of reasonable regularity) then the first approximations of the θ-values and R-values will be sufficiently exact, and the analysis will be completed by the first ten steps. Note, however, that in any case one must then compute the moments from the rotations and deflections by substitution into the general slope-deflection equations.

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[6] "Simplified Methods for Analyses of Multiple-Joint Rigid Frames", by G. A. Maney, M. Am. Soc. C. E., and John E. Goldberg, Jun. Am. Soc. C. E., *Bulletin*, Northwestern Univ., School of Eng., 1932.

[7] Prof., Structural Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[8] See pp. 627–629.

In order to apply the entire fourteen steps of the procedure to the lower four stories of an irregular building having an outline similar to Fig. 2, the writer estimates that one would have to compute approximately two hundred preliminary and final values of rotation, deflection, and moment by substitution into equations varying in complexity from Equation (10*a*) to Equation (12). Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings, has estimated the time required by its proposed method to be 10 min per member,[9] or approximately 5 hr for this structure. This time limit has been reduced by the writer by the use of a simplification of the method proposed by the Sub-Committee. It would seem unlikely that the author's method could be applied within this time limit.

The author's method in common with most methods based directly on simultaneous equations must be classed as mathematical rather than visual. For instance, the various steps of the process cannot be visualized as directly associated with the physical action of the structure. Hence, the operator must depend entirely either on memory or on printed formulas when applying the method. In order to check this statement the writer assigned the analysis of a frame by this method to one of his graduate students. The student followed the outline step by step and obtained a satisfactory analysis. However, finding nothing physical about the process to tie the various steps together, he neither understood nor retained a knowledge of the procedure.

The writer feels that the most valuable part of the author's paper is the discussion of the composite bent. Since the relative amount of load resisted by each bent is measured by its resistance to deflection, it is natural that a slope-deflection method should simplify the problem of the division of load between bents. The writer is inclined to believe that this problem of load distribution has received less than its proper amount of attention from practical designers. There seems to be an unjustifiable belief that distribution of load is of little importance if the total resistance of all bents is adequate. Observe the careless use of diagonal bracing and knuckle connections in adjacent bents. In this same connection, the author mentions torsion in the asymmetrical frame, but offers no solution to the problem. This is one of the most serious problems facing the structural engineer and one which has received but little attention.

The author finally discusses the problem of secondary wind stresses or, more exactly, the effect of column deformations upon wind moments. Clearly, his slope-deflection method is very poorly adapted to the solution of this problem. What excuse could there be for one to use joint movements to calculate joint rotations and lateral deflections when these must then be re-used to calculate moments? The direct procedure is to use joint movements to calculate fixed-end girder moments which are then easily changed into correction moments by the general idea of moment distribution. The original advantage of the slope-deflection method, namely, that there are fewer θ-values and R-values to handle by simultaneous equations than there are joint moments (although the number of equations is the same), is no longer an important advantage when the analysis is performed by successive corrections.

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[9] *Proceedings*, Am. Soc. C. E., February, 1932, p. 218.

The author should be complimented for his attempt to reduce the labor of wind-stress analysis by the slope-deflection method. Despite the fact that he believes that it will consume more time than other methods of successive corrections, the writer realizes that there are many engineers who will find it useful. Particularly, those older engineers who have made wide use of the general slope-deflection method will appreciate the availability of this simpler tool. The paper is also of service in calling the attention of the profession again to the importance of wind-stress calculations in controlling the design of tall buildings. Only by continued emphasis can the less informed members of the profession be made to realize that there is no place to-day for the crude approximations that were the only reasonable tools available a few years ago. The writer was particularly pleased to note the author's remark that his method does not place "too much confidence" in the belief that "points of contraflexure * * * tend to approach the mid-lengths of members." There seems to be an over-confidence among many writers that the ideal of economic design is to accomplish this result.

S. S. GORMAN,[10] M. AM. SOC. C. E. (by letter).—The author states that in wind load distribution "it is usual to assume that the frame carries the entire load." This may be entirely unjustified by the conditions in many instances. In fact, in some cases on which the writer was associated, the walls were so rigid as to carry more than 90% of the lateral loading leaving very slight amounts for the individual bents in the frame. This is a condition which many engineers are prone to ignore, and results in needless waste of material and labor. While it may be considered as an additional factor of safety to carry the entire loading on the frame, this is a waste of the client's money, for which he derives no material benefit. Furthermore, is one justified in using a refined, accurate method of analysis when the frame loading is so small a percentage of the whole, or when the distribution to the individual bents is uncertain? There are numerous occasions when a so-called accurate method is warranted, but the designing engineer must use good judgment in deciding when.

Mr. Goldberg has referred to a "composite bent" in treating the distribution of loading among the parts of the frame. This is an extremely delicate and important process, and, in the writer's opinion, has been too lightly touched upon. The frame of a building, consisting of a number of individual bents, is often unsymmetrical about the center line of the building; that is, the individual bents are dissimilar, or the walls are rarely of equal rigidities. How often are the parallel walls on opposite sides alike? These conditions result in the center of gravity of the loading not coinciding with the center of gravity of the resisting elements, thereby tending to twist the structure about a vertical axis. This torsion should be provided for but often it is not. A very common case is the building on a corner lot, built adjacent to other buildings, with very rigid rear walls without openings therein and with the street-front walls containing numerous openings.

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[10] Senior Bridge Designing Engr., State of California, San Francisco, Calif.

The California earthquake of 1906 made apparent to Western engineers the necessity for more consideration and accuracy than had hitherto existed for determining and providing for lateral deflections, distortions, and stresses in the design of structures. While the fundamental principles for such determinations were understood implicitly, the labor and time required in making the necessary computations by the methods of analyses which were then available, made them impracticable. Many engineers devoted a great amount of time and study in an effort to simplify such computations so that they would be rapid and accurate, but generally without practical success. The contributions of Messrs. Hardy Cross, Wilson and Maney, Nishkian, Grinter, Morris, Goldberg, and many others, that have appeared in recent publications have a value to the structural engineer greater than is generally understood by the profession.

In California the necessity of providing for lateral forces is accentuated by seismic conditions, and shocks which have occurred subsequent to that of 1906 have emphasized this fact. They have been the cause of great increases in earthquake insurance rates on structures and have prompted financial interests which loan money for building purposes to demand earthquake insurance as a part of their consideration for loans. Several of the latter have withdrawn such demands where it could be shown that proper and competent attention had been given to lateral forces in the design and the building had been constructed under adequate inspection and supervision.

Mr. Goldberg's paper seems to be another adaptation of the slope-deflection method very similar to that developed in July, 1930, in the Bureau of Building Inspection, City of San Francisco, by S. M. Cotten, M. Am. Soc. C. E., under the direction of John B. Leonard, M. Am. Soc. C. E.[11] These papers are as accurate as the slope-deflection method and produce identical results therewith when the converging angular rotations are balanced a sufficient number of times. Of course, the author's paper is subject to the same primary, fundamental assumptions as the slope-deflection method, and its accuracy must be gauged by the correctness of these assumptions. Mr. Goldberg's paper is similar in principle to the method of Hardy Cross, M. Am. Soc. C. E., entitled "Analysis of Continuous Frames by Distributing Fixed-End Moments,"[12] except that instead of balancing moments it balances the angles of rotation of the joints of a frame and the deflections thereof, computing the final moments after these are balanced.

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[11] *Western Construction News*, February 10, 1931.

[12] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1.

This paper may be equally adaptable to vertical or to horizontal loading, to symmetrical or unsymmetrical loading or framing, to individual bents or to any number of bents comprising a structure, and to rotation of or translation of the joints of the bent. The simplicity, rapidity, and accuracy which may be obtained in analyzing a frame is indeed surprising when one becomes adept with these operations. In an individual bent one must determine an initial set of values of the angles of rotation of the joints and their corresponding deflections. Balancing is then commenced and the number of repetitions required is dependent upon the accuracy obtained in the initial assumed angles and deflections. One can readily note that a poor selection of these initial values will result in a waste of time; so it behooves one to start with a value as close to the final value as it is possible to predetermine,

The loads carried by any one bent may be apportioned so that the part of the total load at any floor line of the structure will be in inverse proportion to the deflections at that point in the several bents and walls, if each of these bents is considered independent and identically loaded. This is based upon the assumption that the floor acts as a rigid diaphragm. It is obvious that unless the relative rigidity of the several bents and walls is constant for their entire height the ratio of the deflections will vary from floor to floor, and the part of the total load at any floor line, carried by each of the several bents and walls, will also be a variable from floor to floor.

In carrying through the balancing of the parts of the frame, only the relative values of rigidities, angles, and deflections are material; hence for brevity and expediency one may eliminate all constants, if one so desires, from his computations until the parts are finally balanced. These constants are then utilized in obtaining final moments.

One of the advantages of this method is that, within reasonable limits, errors in computations may creep in which will not prevent obtaining the correct solution, but will merely postpone its ultimate attainment. Furthermore, if a change in an angle or a member is necessary, a study will show that the influence in most cases diminishes very rapidly as the angles considered become more remote from each other. One soon learns in such a case, that it is sufficient to consider only that part of the structure and loads which have a material influence on the angle or group considered.

Another advantage is that the angles and deflections corresponding to the several elements of a combined loading may be combined algebraically, and that the moments derived from this summation will be those due to the combined loading. In treating the frame for vertical loading only, initial values of all the angles may be assumed as zero and the balancing proceeded with, obtaining results without excessive expenditures of time.

A great time-saver was presented in the method of the San Francisco Bureau, previously mentioned, which would improve the method presented by Mr. Goldberg very materially, namely, an "equivalent symmetrical bent" of one span for use while analyzing any number of spans in an irregular bent. This single-span symmetrical bent is considered to represent temporarily, the actual bent as to number of stories, story heights, and total distributed rigidities. Due to its symmetry only one column and only one joint at each floor line need be considered. The ΣK of the columns of the equivalent bent equals the ΣK of the columns of the actual bent at any story, and the K-value of the girder of the equivalent bent equals the ΣK of the girders of the actual bent at any floor level. The purpose of this equivalent bent is to determine approximately, and very simply, certain required properties of the actual bent, having which the analysis of the latter is greatly expedited. This consists briefly in balancing the angles of the equivalent bent, obtaining the moments in the ends of the girders, and, with these, calculating the deflection in each story. These moments, deflections, etc., are then transferred to the actual bent and the balancing of the actual angles is commenced. By successive approximations any degree of accuracy can be obtained by carrying the balancing to the extent desired.

During the years 1930 to 1933, inclusive, the method of the San Francisco Bureau has been used by a number of engineers in the making and checking of actual designs of structures, while the San Francisco Bureau has made wide use thereof in the checking of plans submitted for building permits. Due to the parallelism of the two methods, these facts would indicate the practical value of Mr. Goldberg's paper in providing a simple, rapid, and accurate method of analysis for the practicing structural engineer.

While the method of analysis of the rigid structural frame by converging angular approximations apparently has no theoretical limitations, it should not be advocated that it be applied to all possible problems of the rigid frame. Most of the accepted methods of analysis, whether analytical or graphical, have some particular field of usefulness in which they excel, and the choice of method best adapted to the particular problem at hand is one of the major functions of the structural engineer.

Progress is only attained through repeated attempts to better the tools available for one's work. The present paper and many others crystallize the thought of the profession on advancement in the art of rigid frame analysis—and the World must progress!

A. Floris,[13] Esq. (by letter).—In this clearly written paper the author analyzes the wind stresses in multiple-story frames by means of formulas based on the slope-deflection method.

The difficulties that arise in analyzing statically indeterminate structures are due mainly to the necessity of solving the simultaneous elastic equations by the tedious process of successive elimination of the unknowns. From the practical point of view, of course, this is a serious handicap.

In order to overcome this difficulty methods have been developed which are comparatively simple. Instead of using approximate formulas which by repeated corrections can be made to approach the exact values, as the author does in his analysis, it is more convenient to arrange the exact slope-deflection equations in such a way that they can be solved quickly by the principle of iteration (see, for example, the excellent method introduced by Professor G. Unold in which the story equations are combined with the joint equations of the frame so that their number is reduced considerably[14]).

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[13] Civ. Engr., Los Angeles, Calif.

In spite of these more efficient methods the author's analysis is interesting and instructive. It deserves the careful study of those interested in the stress analysis of tall buildings.

JOHN E. GOLDBERG,[15] JUN. AM. SOC. C. E. (by letter).—The discussions of this paper are encouraging and gratifying to the writer. He cannot but feel that they are, at least, expressions of interest in the methods that he has proposed.

Professor Grinter raises the vital question of time required for an analysis. He presupposes that the writer's method requires so much time as to make it comparatively impracticable for ordinary use. On the contrary, the writer believes that it possesses a definite time advantage over methods heretofore proposed. In February, 1932, for purposes of comparison with the method proposed by the Sub-Committee of the Construction Division on Wind-Bracing in Steel Buildings, the writer applied his method to the lower five stories of the Wilson and Maney twenty-story bent, and completed a solution—including calculation of the deflections of the five stories—in 93 min, which indicates a total time of about 370 min for the complete bent. The analysis of the bent by the Sub-Committee's method required about 800 min for moments alone, plus an additional $2\frac{1}{2}$ days for calculation of deflections along two lines of columns, or about 1 800 min for a complete solution, inasmuch as, by any method but a deflection method, it is advisable to calculate deflections along two lines of columns as a check on the accuracy of the solution. At the same time, the writer's solution gave values consistently more accurate than that of the Sub-Committee.

Following the outline prescribed by the writer, a graduate student working under Professor Grinter was enabled to obtain a satisfactory analysis of a particular wind-stress problem to which he was assigned. This fact is gratifying evidence of the inherent accuracy of the method. "However," Professor Grinter states, "finding nothing physical about the process * * *, he neither understood nor retained a knowledge of the procedure." Such an attitude is completely illogical and simply evidences the fact that no sincere effort was made to understand the method, because, of all the phenomena associated with the carrying of load by a single member, or by an integral structure, deformation and deflection alone have a physical and visual significance. Indisputably, slope-deflection methods possess an inherent advantage over other methods on this score. The tangibility is further increased by the method of working (as the writer suggested) directly on a line drawing of the frame under consideration.

Professor Maney points out the basic nature of the theory, and its simple and fundamental relation to convergence. Surely, neither slope-deflection theory nor convergence can be thought of as complicated. By its very nature, the proposed method possesses a physical tangibility that is indisputable.

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[14] "Die praktische Berechnung der Stahlskelettrahmen" von Georg Unold, Berlin, 1933.

[15] Chicago, Ill.

Mr. Gorman places his finger on the crux of the matter. He writes, "the simplicity, rapidity, and accuracy which may be obtained in analyzing a frame is indeed surprising when one becomes adept with these operations."

Professor Grinter feels that the derivation of the formulas for the initial values of deflection is too long. There is a much shorter method of attack, but the writer presented the longer, more formal, derivation chiefly because it suggests the method by which the formulas for the initial value of the first-story joint rotation is obtained from fundamental considerations. However, it may be well, at this point, to outline the shorter derivation.

Assume, as in the paper, that all joints of the nth story of the bent under consideration, as well as of the mth and oth stories, have the same angular rotation, θ'_n. Then, by Equation (3), the moment at each end of each beam or girder of the nth story is, $- K_{Gn}\ (3\ \theta'_n)$, and the sum of the end moments for all the beams or girders of the nth story of the bent will be, $- \Sigma\, K_{Gn}\ (6\ \theta'_n)$. The sum of the column end moments of the nth story is M_n, and that of the oth story is M_o. Since at each joint of the nth story, $\Sigma_m = 0$, the same law must hold for all joints of the nth story taken together. Thus, $\frac{M_n}{2} + \frac{M_o}{2} - \Sigma\, K_{Gn}\ (6\ \theta'_n) = 0$; and, solving for θ'_n,

$$\theta'_n = \frac{M_n + M_o}{12\ \Sigma\ K_{Gn}} \quad \text{..........................(16)}$$

Direct substitution of θ' into the bent equation gives an expression for the initial value of the corresponding R'.

In presenting his conception of the composite bent, the writer expressly pointed out that it would find its most ready application in connection with the analysis of symmetrical and practically symmetrical buildings. However, Professor Grinter feels that it is a commendable contribution and he goes on to state that, "* * * it is natural that a slope-deflection method should simplify the problem of the division of load between bents." Clearly, his statement points out an inherently advantageous characteristic of the method which the writer has proposed.

Mr. Gorman suggests a method for the analysis of symmetrical buildings which is so admirably straightforward that, no doubt, its value will be tested in the near future by application to actual problems. He objects to the statement that "it is usual to assume that the frame carries the entire [wind] load," and he cites an example in which the walls of a particular building carried about 90% of the lateral loading, leaving only a small amount for the frame. Obviously, such a structure as this should not be analyzed simply as a frame. The walls themselves should be designed and analyzed as the primary load-carrying units, and the frame should be considered more or less as a secondary effect, allocating to it a proportion of the total load based on some rational relationship between the action of the frame and the action of the walls; that is, deflections, etc.

Mr. Gorman suggests the temporary substitution of an equivalent single-span symmetrical bent for the actual bent. For the usual type of problem, requiring only a few cycles of approximations, exactly the same purpose is served simply by using the formulas of the paper for the initial values of joint rotation and story deflection, θ' and R'.

Mr. Floris refers to the approximate equations that the writer uses in the analysis. Possibly, the writer has misunderstood Mr. Floris' reference, but it is well to emphasize the fact that only the equations for θ' and R' are approximate formulas; all other equations used in the analysis are the correct slope-deflection equations.

Personal preference, no doubt, will be an important element in the choice of a method of wind-stress analysis. However, the attributes of the various methods should not be overlooked completely. Accuracy, is essential, of course. Workability has been added to correct basic theory to so great an extent that the use of methods of doubtful accuracy and undependable results no longer can be justified. Furthermore, a method that offers speed as well as the certainty of accuracy is to be preferred, naturally; and, again, wide scope and broad applicability should be important considerations. For example, a method that offers a rational means of analyzing complete buildings is preferable to a method of limited application.

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Paper No. 1879

ON THE BEHAVIOR OF SIPHONS

BY J. C. STEVENS,[1] M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. IVAN M. NELIDOV, HERBERT H. WHEATON, AND J. C. STEVENS.

SYNOPSIS

Results of field tests of three siphons having nominal capacities of 100, 250, and 500 sec-ft, respectively, are presented in this paper. Important effects of cavitation are disclosed. Pressure gradients as well as pseudo, and true, energy lines are given. A simplified conception of the action of siphons is presented, involving an analogy to free flow and submerged orifices. There is also offered a method of comparing siphons by a standardized coefficient of flow and a standard expression for the efficiency of a siphon.

THE LEABURG SIPHONS

The Leaburg Hydro-Electric Plant, of the municipal power system of the City of Eugene, Ore., consists essentially of a dam in the McKenzie River that raises the water 20 ft for diversion into a power canal, 5 miles long, with a capacity of 2 200 sec-ft. The canal ends in a forebay from which water under a 90-ft head is passed through 12-ft concrete penstocks and 10 000 hp water-wheels, and returned to the river through an excavated tail-race.

The wasteway from the forebay consists of seven siphons, having an aggregate capacity of 3 000 sec-ft, sufficient to relieve the canal should a sudden stoppage of the turbines occur under full flow and during a period when the areas above the canal are draining heavily into it.

Since the plant has only one generating unit installed at present (1933), three of the channels leading from the siphons have not been completed although they are available for use in an emergency.

NOTE.—The subject of "Losses in Siphon Spillways" is one of those selected by the Special Committee on Irrigation Hydraulics for study and research. This paper was presented to the Committee by its author, and the Committee recommended its publication by the Society. Published in August, 1933, *Proceedings*.

[1] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

Fig. 1 shows the inlet to the siphons. In Fig. 2, the water seen in the outlet channel of Siphon No. 1, is from the sluice-gate shown in Fig. 1. The trash from the racks may be sluiced to the river in the channel entering the outlet from Siphon No. 7 at the left.

Three sizes of siphons were used in order to obtain elasticity of performance. Siphon No. 7 (Fig. 2), was designed for a nominal capacity of 100 sec-ft; Siphon No. 6, for 250 sec-ft; and the other five, for 500 sec-ft each.

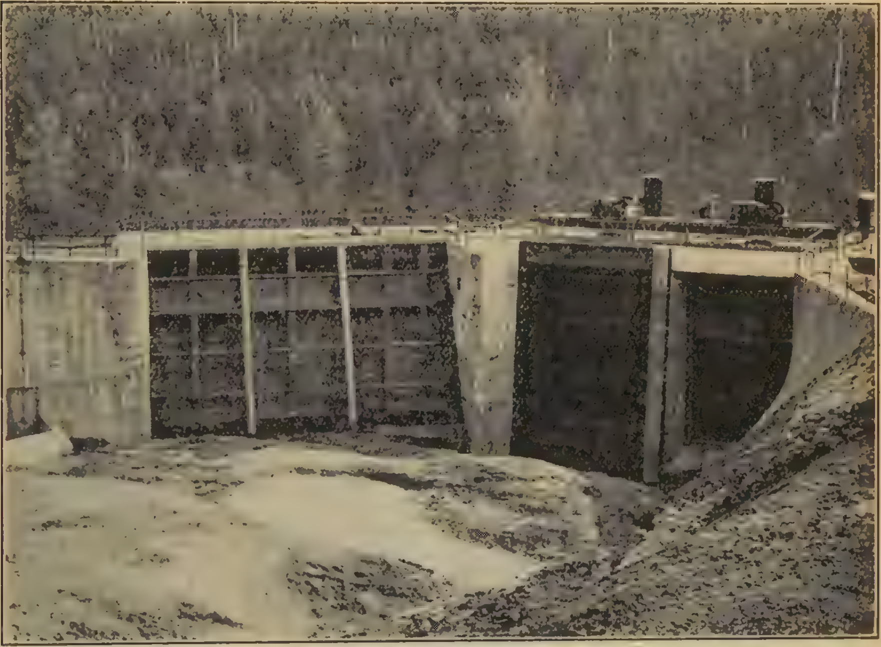


FIG. 1.—FOREBAY STRUCTURE; AT LEFT IS INLET TO SEVEN SIPHONS; AT RIGHT, INLET TO PENSTOCKS.

Fig. 3 and Table 1 show the essential dimensions of the siphons measured after they were completed. The primer consists of a steel gate held between guides, which can be set with its lower edge at or above the crest of the siphons.

The partition walls at the immediate outlet channels of the siphons (Fig. 2), were left unfinished until after tests were made, in order that the siphon barrels could be extended, or the outlets nozzled, or any other alterations made that the tests might indicate as desirable to improve the conditions of flow.

In order to determine the performance of each size of siphon, a set of four piezometers was installed in each of six sections in Siphons Nos. 5, 6, and 7. The sections chosen were relatively the same for each siphon, as shown in Figs. 3(*a*), 3(*b*), and 3(*c*). At each section the piezometers are disposed as shown in Fig. 3. Pipes were brought out to the face of the concrete and connected to a header with stop-cocks so that the pressure in each piezometer could be obtained on a single manometer.

On the inside face of the siphon barrel the pipe was laid flush with the forms. After the forms were removed, the face was rubbed down until the metal and concrete were smooth and in a uniform plane at right angles to the direction of flow.

The crests of the siphons are all at the same elevation, 734.0, which is the normal operating level of the forebay. The priming gate on any siphon may be raised to put that one out of service. When so raised, water flows over the crest as if it were an open spillway. By adjusting the elevation of the lower edge of the primer gate, the sequence of starting and stopping of siphons can be controlled.

There is a recess on three sides of the barrel above Section 3 in each siphon, the purpose of which is to admit air under the nappe when a thin sheet of water is flowing over the crest. This equalizes the pressure within the siphon on both sides of the nappe and permits the water to leave the concrete and fall across the barrel, accelerating the removal of air.

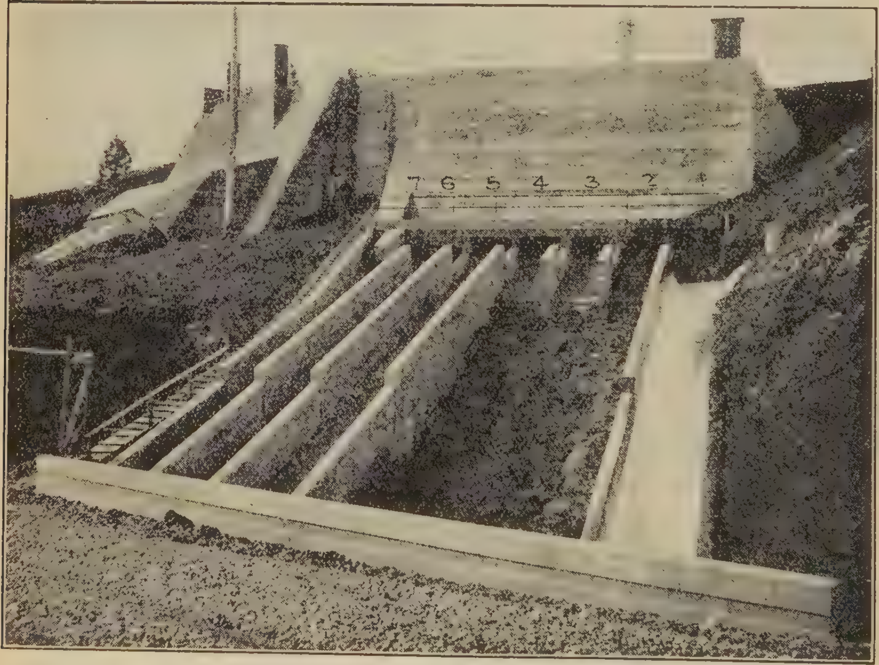
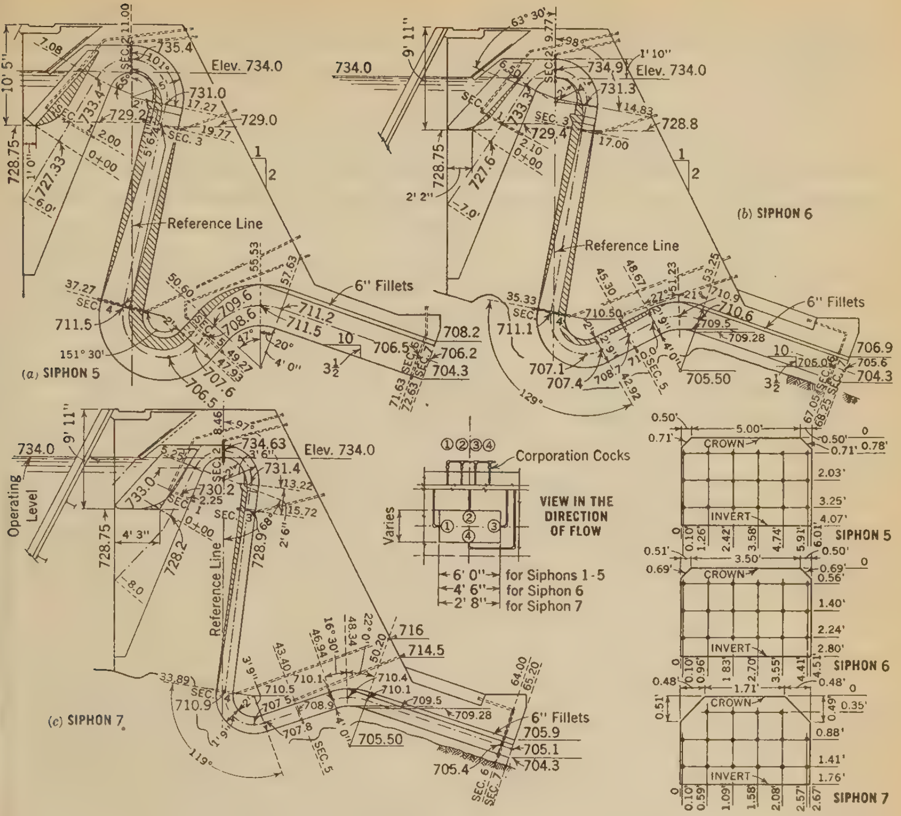


FIG. 2.—OUTLET OF LEABURG SIPHONS.

BEHAVOIR IN OPERATION

The plant and siphons had been in operation more than two years before the tests were made. In operation the siphons functioned better than was anticipated. Experience has shown that if the priming gates of all siphons are at their lowest point (Elevation 734.0), the siphons will prime in the order of their size when water rises in the forebay; that is, Siphon No. 7 primes first with a nominal head of 0.2 to 0.3 ft over the crest; Siphon No. 6 primes next with a head of 0.3 to 0.4 ft. The order of priming of the large siphons is somewhat erratic if all priming gates are down. For days,



(a) SIPHON 5
(b) SIPHON 6
(c) SIPHON 7
Elev. 734.0
734.0
Reference Line
6'' Fillets
10' 5''
9' 11''
728.75
735.4
731.0
729.0
729.2
711.5
711.2
708.2
706.5
706.2
704.3
151° 30'
734.9
731.3
728.8
729.4
711.1
710.6
706.9
705.6
705.50
129°
734.63
731.4
730.2
728.9
716
714.5
710.9
705.9
705.1
705.4
119°
Operating Level
①②③④ Corporation Cocks
VIEW IN THE DIRECTION OF FLOW
Varies
6' 0'' for Siphons 1-5
4' 6'' for Siphon 6
2' 8'' for Siphon 7
CROWN
INVERT
SIPHON 5
SIPHON 6
SIPHON 7
5.00'
3.50'
1.71'

FIG. 3.—ESSENTIAL DIMENSIONS OF SIPHONS NOS. 5, 6, AND 7. SECTION 7 OF EACH SIPHON ENLARGED TO SHOW PITOT TUBE POSITIONS.

Siphon No. 5 will be the next in order, starting with a head of 0.4 ft when, with no apparent reason, Siphon No. 1 will start under the same head instead of Siphon No. 5. The priming gates on Siphons Nos. 2, 3, and 4 are kept raised so that they would only come into action as an emergency spillway.

TABLE 1.—ESSENTIAL DIMENSIONS,* SIPHONS NOS. 5, 6, AND 7.

Section	SIPHON NO. 5			SIPHON NO. 6			SIPHON NO. 7		
	Width, in feet	Height, in feet	Area, in square feet	Width, in feet	Height, in feet	Area, in square feet	Width, in feet	Height, in feet	Area, in square feet
1 primer........	....	....	9.0	....	....	6.75	....	....	3.3
1 barrel........	....	....	30.8	....	....	18.00	....	....	5.7
Total.......	6.03	5.01	39.8	4.50	4.01	24.75	2.66	2.14	9.0
2............	6.03	2.75	16.5	4.52	1.77	8.00	2.68	1.26	3.43
3............	6.01	3.18	19.1	4.53	2.12	9.60	2.69	1.58	4.20
4............	6.02	4.09	24.6	4.53	2.78	12.50	2.69	1.75	4.70
5............	6.00	4.02	24.1	4.52	2.78	12.55	2.69	1.77	4.77
6............	6.01	4.07	24.2	4.55	2.80	12.07	2.67	1.76	4.46
7............	6.01	4.07	24.2	4.51	2.80	12.42	2.67	1.76	4.43

* All sections rectangular, except Sections 6 and 7 (see Fig. 3).

During the operation of the plant the quantity diverted at the dam is so regulated that Siphon No. 7 runs almost continuously and Siphon No. 6 operates at times of rejecting load only. The largest siphon seldom comes into action.

A most gratifying behavior of the siphons is the facility with which the flow through them automatically adjusts itself to the surplus inflow to the forebay. As the level of the water in the forebay drops below the edge of the priming gate, air is sucked into the barrel in rhythmic gulps at intervals of about 1 sec. This mixture of air reduces the flow, which arrests the drop of the water in the forebay. Once it has been started, the flow through any siphon may thus diminish to a small part of its normal capacity without breaking its siphonic action.

The result is that almost regardless of the flow into the forebay, or of that taken by the turbine, the water level in the forebay rarely varies more than 0.10 ft above or below its normal operating level of 734.0. These siphons are true "auto-leveling siphons," as they have been called in Italy.[2]

One of the chief concerns in the tests was to be sure the siphon under test was not sucking air. This condition could be met if the water level in the forebay was kept high enough to insure the lower edge of the priming gate being substantially submerged.

METHODS OF TESTING

Tests of the performance of the siphons were made from November 19 to 22, 1931. The flow was measured by a Pitot tube at the outlet of the siphon barrel. Guides were built over the outlet so that the nozzle of the Pitot tube could be held in any position in the verticals shown in Section 7 of each siphon (Fig. 3). These verticals are spaced 0.10 ft from each side and then one-fifth of the remaining width.

In general, velocities were observed at two-tenths, five-tenths, and eight-tenths, of the depth of the barrel. On Siphons Nos. 6 and 7, those at two-tenths the depth at each side vertical were omitted on account of the fillets.

The Pitot nozzle proper was 3 in. long with a $\frac{1}{8}$-in. orifice. It was held in the plane of the outlet of the barrel and parallel to the direction of flow by a length of $1\frac{1}{2}$-in. extra strong pipe which terminated in a connection for, and was joined to, a length of small hose. The other end of the hose terminated in a short glass tube. This glass tube was held beside a level rod and the water level in the tube was recorded at each position of the Pitot nozzle. The elevation of the water in the tube above the water surface at the outlet of the siphon barrel is the velocity head of the water velocity at the position of the nozzle of the Pitot tube. A coefficient of 98% was used for this nozzle. To avoid errors from accumulated air, the hose line was "bled" constantly, except at the instant of reading.

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[2] "Experimental Research on Auto-Leveling Siphons," Second Report, by Prof. Guilio de Marchi and Ing. Mario Marchetti, *L'Energia Elettrica*, November 1931; *Technical Memorandum No. 285*, translation from the Italian by the Office of Education, Washington, D. C.

The flow in Siphon No. 7 was first measured by observing velocity heads on a mercury manometer, and, later, by direct observations, in feet of water, as described. The flows obtained were 119 sec-ft to the nearest unit in both cases, although the individual velocities varied somewhat (see Fig. 4(*a*)).

The vertical velocity curves are of considerable interest. Fig. 4 shows them for each siphon outlet. The mean velocity in each vertical was determined by planimeter, and these means were used to form a horizontal velocity curve (see Fig. 5). The arrows and figures show the position and velocity of the mean thread in each vertical. The variations in velocities from the two measurements on Siphon No. 7 are seen to be compensating. The mass of flowing water oscillated rhythmically slightly from side to side, due perhaps to conditions at the inlet.

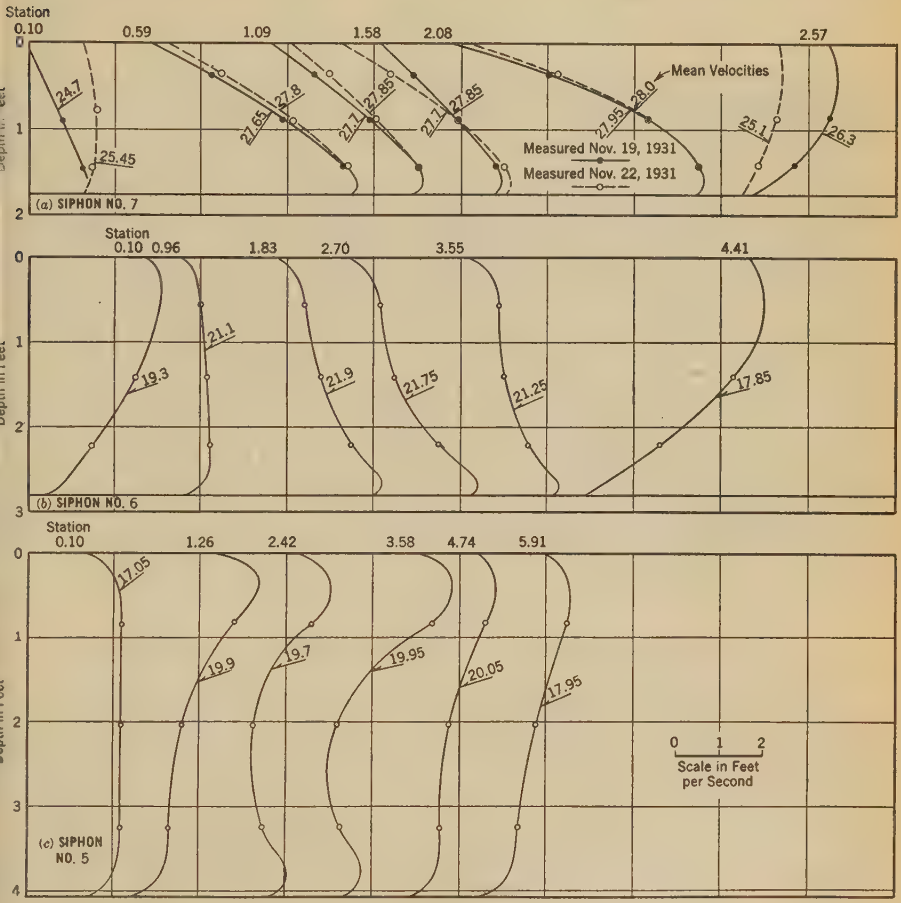
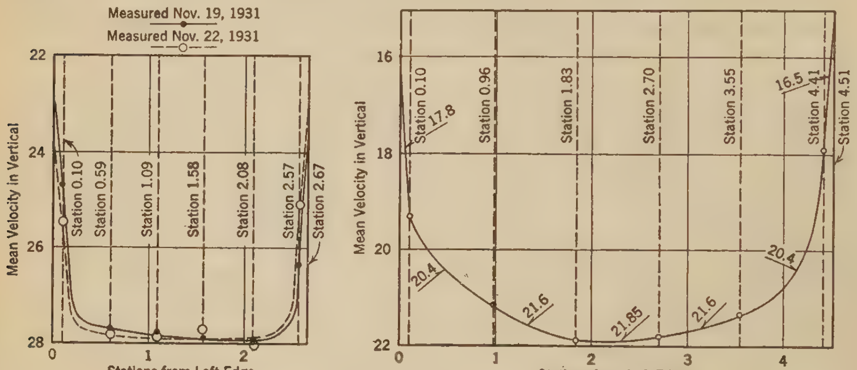


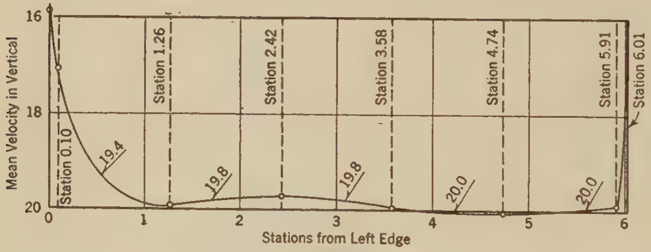
FIG. 4.—VERTICAL VELOCITY CURVES.

The variation in the shape of the vertical velocity curves between siphons shows strikingly the effect of the curves in the siphons. The mass swings from one face to the other as it passes through the siphon. The position of the mass of maximum velocity is dependent entirely upon the flow and the proximity of the plane of measurement to the curve up stream from it.



(*a*) SIPHON NO. 7

(*b*) SIPHON NO. 6



(*c*) SIPHON NO. 5

FIG. 5.—HORIZONTAL DISTRIBUTION OF MEAN VELOCITIES IN THE VERTICAL.

During tests, the water issued from the barrel in a solid prism of clear water very similar to a fire stream at the nozzle end. Nevertheless, it is practically certain that Siphon No. 5 was carrying considerable entrained air. The measured flow was 470 sec-ft, whereas it is believed to be capable of discharging 500 sec-ft, or more. The I-beam supporting the rack caused a drop in the water surface just before it enters the primer. This was more noticeable with the large siphons than with either of the others. Air was seen to be sucked in small whirlpools during the test. The outlet, however, showed no evidence of air.

Pressures were read on a mercury U-tube, or manometer. One manometer was fastened to the hand-rail at the top of the siphon block and another to the rail at the deck over the outlets. Each manometer commanded three sections. Pipe fittings terminated in a $\frac{3}{8}$-in. gauge-cock. The hose used was $\frac{1}{2}$-in. round rubber packing, having a $\frac{1}{4}$-in. hole which admirably fits the nipple end of this particular gauge-cock. This packing makes an excellent hose for both positive and negative pressures. It will stand a pressure of more than 100 lb without blowing the packing off the nipple, so that clamps or wiring are rarely required at joints. Fig. 6 shows two sizes of manometers used in the tests. By turning the proper stop-cocks, the pressure in any one piezometer or in the header at each of three sections could be read from a single manometer.

All pressures in the siphons, with the exception of some of those in Sections 1 and 6, were negative so that the relative elevations of manometer and piezometer were not a factor. Where positive pressures were encountered, air was forced into the rubber hose with a tire pump, expelling the water in the piezometer pipe and giving the true pressure on the mercury column.

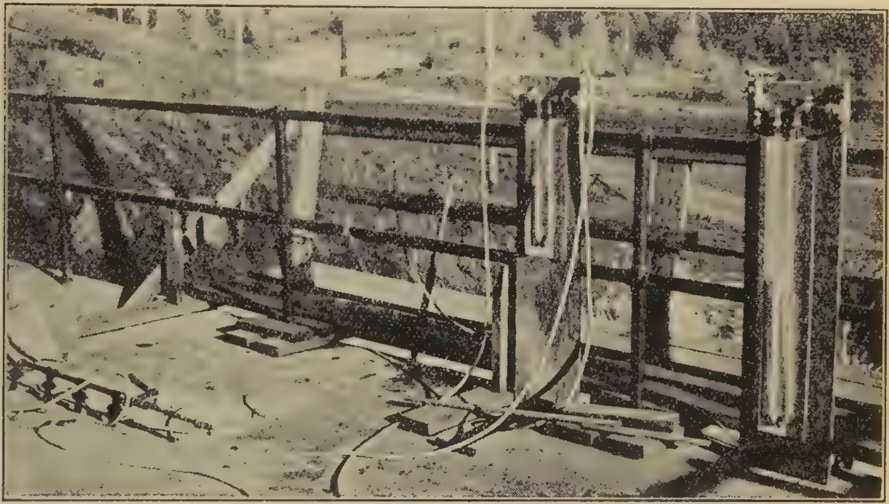


FIG. 6.—MERCURY U-TUBES AND HOSE CONNECTIONS TO PIEZOMETERS ON LOWER DECK OF SIPHON BLOCK.

Oscillations in the water and mercury columns were troublesome. They were dampened by inserting a by-pass fitting in the hose line which was closed by a plug through which a $\frac{1}{64}$-in. hole was drilled. Cocks were included so that the open leg could be used for testing and "bleeding" and the by-pass for reading pressures. This device proved to be very effective and insured a "live" hose line at all times.

RESULTS OF THE TEST

Table 2 gives for each siphon the pressures obtained from each piezometer. Those marked "Header" are the pressures from all piezometers in one section when all are open to the header. The elevation of each piezometer orifice (where the pipe meets the barrel of the siphon) is given, with the elevations of the pressure lines.

Table 3 is a summary of the essential data obtained by the tests. The pressures in Section 6 are omitted on account of uncertainties due to oscillations and to near-zero pressures. The portion of the flow that passed through the primer has been estimated. Elevations of the pressure line and of the energy lines, *A* and *B*, are in absolute values; that is, an atmospheric head of 33 ft has been included.

TABLE 2.—PRESSURE READINGS OBTAINED FROM PIEZOMETERS AT EACH SIPHON

Section (1)	Piezometer No. (2)	SIPHON NO. 7			SIPHON NO. 6			SIPHON NO. 5		
		Elevation of piezometer orifice (3)	Pressure head (4)	Elevation of pressure line (5)	Elevation of piezometer orifice (6)	Pressure head (7)	Elevation of pressure line (8)	Elevation of piezometer orifice (9)	Pressure head (10)	Elevation of pressure line (11)
Forebay.....	....	.....	......	767.3	.....	......	767.3	.....	......	767.6
1...........	1......	730.2	—2.8	760.4	729.4	0	762.4	729.2	—1.5	760.7
	2......	730.7	—2.9	760.8	730.4	—0.2	763.2	730.9	—3.8	760.1
	3......	730.2	—2.8	760.4	729.4	—0.2	762.2	729.2	—1.1	761.1
	4......	729.7	—2.8	759.9	728.4	—0.2	761.2	727.5	—0.2	760.3
	Header.	730.2	—3.0	760.2	729.4	—0.2	762.2	729.2	—1.6	760.6
2...........	1......	734.6	—23.8	743.8	734.9	—13.8	754.1	735.4	—14.5	753.9
	2......	735.2	—17.0	751.2	735.8	—9.6	759.2	736.8	—10.9	758.8
	3......	734.6	—25.4	742.2	734.9	—14.6	753.3	735.4	—14.1	754.3
	4......	734.0	—30.9	736.1	734.0	—24.3	742.7	734.0	—28.8	738.2
	Header.	734.6	—25.6	742.0	734.9	—15.4	752.5	735.4	—16.2	752.2
3...........	1......	728.9	—28.5	733.4	728.9	—32.0	729.9	729.0	—28.7	733.3
	2......	728.8	—28.5	733.3	728.6	—32.5	729.1	728.5	—28.0	733.5
	3......	728.9	—31.0	730.9	728.9	—32.4	729.5	729.0	—31.5	730.5
	4......	729.0	—30.3	731.7	729.2	—32.4	729.8	729.5	—30.7	731.8
	Header.	728.9	—31.1	730.8	728.9	—32.4	729.5	729.0	—30.4	731.6
4...........	1......	710.9	0	743.9	711.1	—2.1	742.0	711.5	—1.4	743.1
	2......	710.8	—2.1	741.7	710.8	—2.2	741.6	710.9	—7.5	736.4
	3......	710.9	+0.3	744.2	711.1	—2.2	741.9	711.5	—1.1	743.4
	4......	711.0	+0.8	744.8	711.4	—2.2	742.2	712.1	0	745.1
	Header.	710.9	—1.2	742.7	711.1	—2.2	741.9	711.5	—5.6	738.9
5...........	1......	708.9	—0.7	741.2	708.7	—1.6	740.1	708.6	—1.8	739.8
	2......	709.7	—0.4	742.3	710.0	—2.9	740.1	710.1	—3.3	739.8
	3......	708.9	—0.3	741.6	708.7	—2.6	739.1	708.6	—3.3	738.3
	4......	708.1	—0.1	741.0	707.4	—2.6	737.8	707.1	—1.1	739.0
	Header.	708.9	—0.6	741.3	708.7	—2.8	738.9	708.6	—3.7	737.9
6...........	1......	705.4	+0.1	738.5	706.0	—0.9	738.1	706.5	+0.2	739.7
	2......	706.2	+0.1	739.3	707.3	—1.2	739.1	708.4	—0.7	740.7
	3......	705.4	+0.1	738.1	706.0	—1.1	737.9	706.5	+0.4	739.9
	4......	704.6	+0.1	737.7	704.7	—1.1	736.6	704.6	+1.6	739.2
	Header.	705.4	+0.1	738.5	706.0	—1.1	737.9	706.5	+0.9	740.4
Outlet.......	....	.....	......	738.9	.....	......	739.9	.....	......	741.2

Piezometers Nos. 1 and 3 are on the center lines of the siphon barrels at the sides, while Piezometers Nos. 2 and 4 are at the crowns and inverts, respectively (Fig. 3(*d*)). Theoretically, the average of the pressure-line elevations from Piezometers Nos. 1 and 3 should be practically the same as the averages of Piezometers Nos. 2 and 4; also, the average of Piezometers Nos. 1, 2, 3, and 4 should be nearly the same as when all are open to the same header, as indicated under "Header" in Tables 2 and 3. An examination of the elevations in Table 2 will show that this condition is approximately fulfilled. Such departures of the "header" readings from the average of all piezometers as exist may be readily attributed to oscillations.

DISCUSSION OF RESULTS

If the flow be divided by the area of each section (Column (3), Table 3) and the resulting velocity head be added to the pressure line (Column (6), Table 3), the elevation of a point on Line *A* (Figs. 7, 8, and 9) is obtained. Line *A* may be thought of as a false energy line, because it is seen to increase above some preceding value. A true energy line must drop continually in the direction of flow.

TABLE 3.—SUMMARY OF ESSENTIAL DATA

Section (1)	Station (2)	Area (3)	Velocity, V_a (4)	Velocity head, h_a (5)	Elevation of pressure line (6)	Elevation of Line *A* (7)	Elevation of Energy Line *B* (8)	Velocity head, h_b (9)	Velocity, V_b (10)	Useful area, *a* (11)
SIPHON NO. 7: FLOW, 119 SECOND-FEET (19 CUBIC FEET PER SECOND THROUGH PRIMER)										
Forebay	.....	......	......	......	767.3	767.3	767.3	0	......	......
1	2.25	9.00	13.2	2.7	760.2	762.9	766.4	6.2	20.0	5.0
2	8.46	3.43	34.6	18.6	742.0	760.6	760.6	18.6	34.6	3.43
3	15.72	4.23	28.1	12.3	730.8	743.1	758.7	28.2	42.6	2.80
4	33.89	4.70	25.2	9.9	742.7	752.6	751.8	9.9	25.2	4.70
5	43.40	4.77	25.0	9.7	741.3	751.0	751.0	9.7	25.0	4.77
Outlet	65.20	4.43	26.9	11.2	738.9	750.1	750.1	11.2	26.9	4.43
SIPHON NO. 6: FLOW, 250 SECOND-FEET (35 CUBIC FEET PER SECOND THROUGH PRIMER)										
Forebay	.....	......	......	......	767.3	767.3	767.3	0	......	......
1	2.10	24.75	10.1	1.6	762.2	763.8	766.8	4.6	17.2	14.2
2	9.77	8.00	31.3	15.2	752.5	767.7	765.0	12.5	28.4	8.8
3	17.00	9.60	26.0	10.5	729.5	740.0	759.6	30.1	44.0	5.7
4	35.33	12.50	20.0	6.2	741.9	747.1	752.0	10.1	25.5	9.8
5	45.30	12.55	19.9	6.2	738.9	745.1	749.4	10.5	26.0	9.6
Outlet	68.25	12.42	20.1	6.3	739.9	746.2	746.2	6.3	20.1	12.4
SIPHON NO. 5: FLOW, 470 SECOND-FEET (60 CUBIC FEET PER SECOND THROUGH PRIMER)										
Forebay	.....	......	......	......	767.6	767.6	767.6	0.	......	......
1	2.00	39.8	11.8	2.2	760.6	762.8	766.6	6.7	20.8	19.7
2	11.00	16.5	28.5	12.6	752.2	764.8	764.8	12.6	28.5	16.5
3	19.77	19.1	24.6	9.4	731.6	741.0	760.3	28.7	43.0	10.9
4	37.27	24.6	19.1	5.7	738.9	744.6	751.4	12.6	28.5	16.5
5	49.27	24.6	19.1	5.7	737.9	743.6	748.8	10.9	26.5	17.7
Outlet	72.83	24.2	19.4	5.9	741.2	747.1	747.1	5.9	19.4	24.2

The reason Line *A* does not drop continually is because the mass of moving water fills only a part of the barrel of the siphon, the remaining part being occupied by eddies; that is, cavitation exists within the siphon barrel. The elevation of the energy line at the inlet is that of the forebay level plus he atmospheric head of 33 ft; at the outlet, it is that of the crown of the arrel (open water surface) plus the exit velocity head, plus 33 ft. A straight ne drawn between these points would not be seriously inaccurate, but ne line marked, "Energy Line *B*" (Figs. 7, 8, and 9), is believed to represent the conditions more accurately.

The differences between the elevations of Line *B* and those of the pressure ne are the velocity heads from which the velocity of the mass of flowing ater is obtained. Dividing the flow by the velocities thus obtained gives e "Useful Area *a*" for each section (Column (11), Table 3). Maintaing the same width of barrel, the amount the actual areas are reduced to tain "useful areas," is shown by hatching on Fig. 3.

The hatched areas represent the parts of the siphon barrels occupied by dies. Showing them alternately on the crown and invert may not repre-

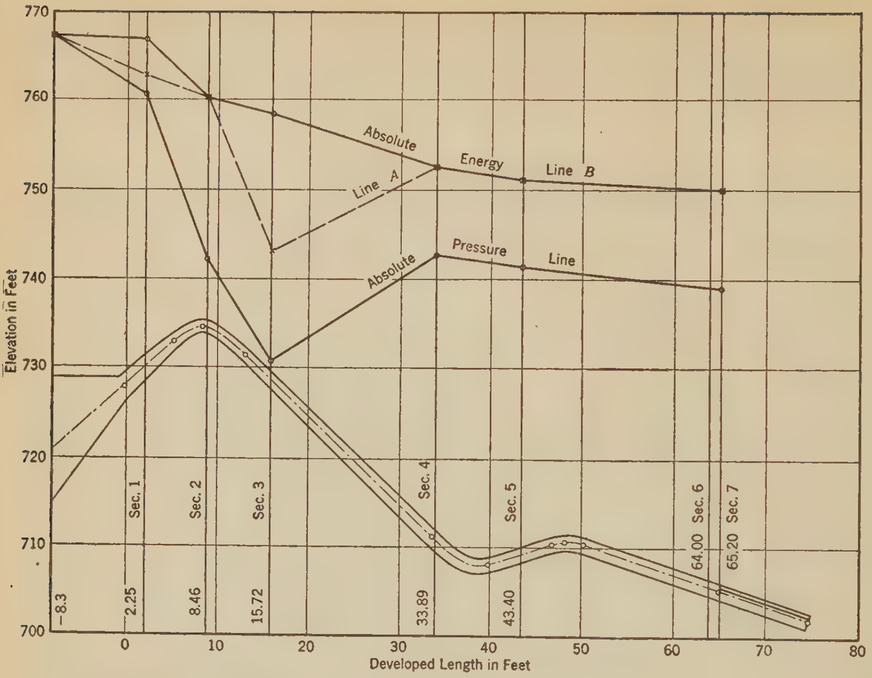


FIG. 7.—PRESSURE AND ENERGY GRADIENTS, SIPHON NO. 7.

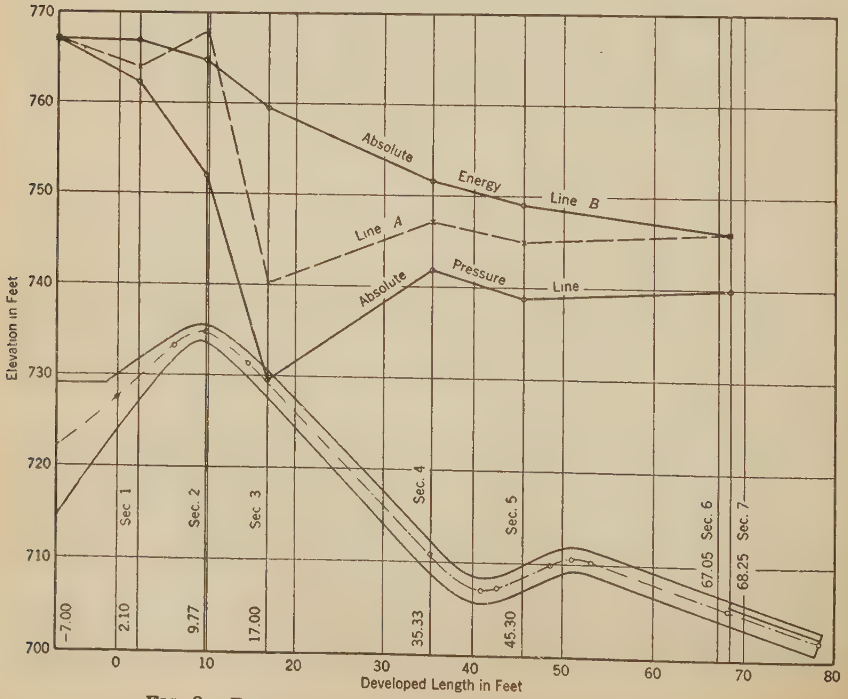


FIG. 8.—PRESSURE AND ENERGY GRADIENTS, SIPHON NO. 6.

sent what actually occurs. For instance, it is entirely possible that the mass of flowing water may swing from crown to invert several times in the long vertical leg, instead of only once as shown.

The siphon barrels are converging from inlet to the summit at Section 2, diverging from Section 2 to Section 4. From the latter they are of constant area to the outlet, except for the fillets in the corners on the outlet leg which have the effect of slightly nozzling the outlet. The tangent of the angle of total dilation from Section 2 to Section 4 is 0.019; that is, if the barrel was straightened and the crown and invert were extended they would meet in an apex angle of 1°5′. The dilation from Section 2 to Section 3, however, is at a greater rate, the tangent of the apex angle being 0.044, corresponding to 2° 32′.

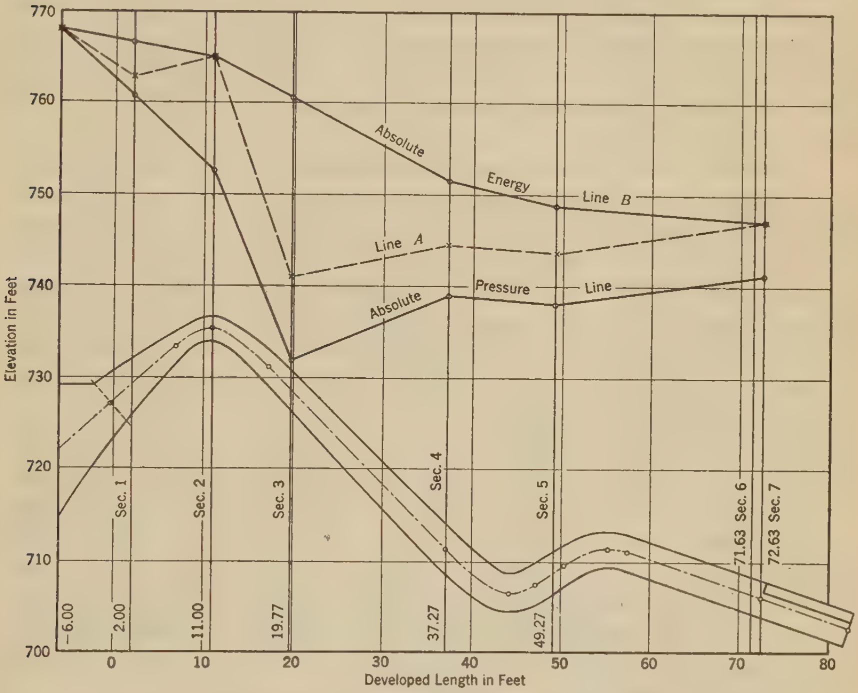


FIG. 9.—PRESSURE AND ENERGY GRADIENTS, SIPHON NO. 5.

Such experiments[3] as have been made on comparatively small pipes indicate that a 6° dilation angle gives the minimum loss. These experimental losses include both friction and eddy losses. For a given enlargement the friction loss increases as the dilation angle decreases on account of the greater length of conduit included in the conversion. The eddy losses, on the other hand, must diminish with the dilation angle. The sum of eddy and friction losses, therefore, has a minimum value which for such experiments as have been made appears to be between 6 and 10 degrees. If eddy losses are divorced from friction losses, however, it is practically certain that the smaller the apex angle the smaller the eddy loss, becoming zero simultaneously with the apex angle.

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[3] "The Conversion of Kinetic to Pressure Energy in the Flow of Water Through Passages Having Divergent Boundaries," by A. H. Gibson, University Coll., Dundee, Scotland, *Engineering* (London), February 16, 1912.

It appears from the foregoing data that the dilation angle of $2\frac{1}{2}$° between Sections 2 and 3 is too great to maintain the barrel full of flowing water under the high velocities obtaining in these siphons. The mass flowing from Section 2 continued to contract, reaching its minimum section near Section 3. Thereafter, it gradually expanded independent of the dilation of the barrel.

That Section 3 was the one of maximum velocity was evidenced during the test of Siphon No. 6. Notice that the pressure head in Section 3 is practically one atmosphere. While the test was under way a small column of water was shot from Section 3 through about 15 ft of $\frac{1}{8}$-in. hose on to the high leg of the mercury column. This was puzzling at first until it was realized that the water column in the barrel separated momentarily under the nearly complete vacuum, and the impact of joining again was sufficient to produce the phenomenon noted. This was always noticeable when Siphon No. 6 was running, by rhythmic impacts.

At the inlet the flow conditions are complicated by the division of flow, part flowing in the barrel proper and part through the primer. The estimated division of flow is given in Table 3. On entering the siphon barrel the water flows nearly horizontal, particularly at the crown of Station 0.00. This doubtless prevents the barrel from filling. Deducting the hatched area (Fig. 3) leaves what probably is a more desirable shape. The water entering the primer joins that entering the barrel at the summit, which has the effect of producing eddies and, no doubt, accounts for some of the anomalous pressure readings obtained.

Siphons are usually compared on the basis of their coefficients of flow, which is expressed by the formula,

$$C = \frac{Q}{a\sqrt{2gH}} \quad \dots\dots\dots\dots\dots\dots\dots\dots (1)$$

in which, a is the transverse area, and H is the total head. Some writers base this coefficient on the throat or summit area and others on the outlet area. Table 4 shows the coefficients obtained under both assumptions for the siphons in question.

The variation in these coefficients and the possibility of manipulating them by varying the areas of the siphon put emphasis on the necessity of a standardization of nomenclature. Before offering suggestions to accomplish this it is well to review briefly certain theoretical considerations regarding siphons in general.

TABLE 4.—Coefficient of Flow

Description	Siphon No. 7	Siphon No. 6	Siphon No. 5
Flow, in cubic feet per second, Q	119	250	470
Area at summit, in square feet, a_2	3.43	8.00	16.5
Total head, in feet, H	28.4	27.4	26.4
$\sqrt{2gH}$, in feet per second, V_H	42.7	42.0	41.2
$a_2\ V_H$, in cubic feet per second, Q_2	146	336	680
$\frac{Q}{Q_2} = C_2$	0.82	0.75	0.69
Area at outlet, in square feet, a_7	4.43	12.42	24.20
$a_7\ V_H$, in cubic feet per second, Q_7	189	521	998
$\frac{Q}{Q_7} = C_7$	0.63	0.48	0.47

Theoretical Analysis

The writer has used certain terms that should be defined before an intelligent discussion can be undertaken; namely:

Summit.—The highest transverse section of the barrel of the siphon at right angles to the direction of flow.

Invert.—The side of the barrel corresponding to the bottom at the summit.

Crown.—The side opposite the invert.

Static Head.—The water head on any point in the barrel of the siphon; that is, the difference in elevations of the point in question and the water surface at the inlet.

Atmospheric Head.—The head of water corresponding to a pressure of one atmosphere (33 ft at Leaburg).

Absolute Head.—The static head plus the atmospheric head.

Siphon Head.—The static head on the water surface at the outlet; that is, the difference in elevations of the water surfaces at inlet and outlet.

Absolute Pressure Head.—If from the absolute head at each point in the siphon there is deducted the energy losses to that point and the velocity head at the point, the remainder is the absolute pressure head on that point. It is the pressure shown by a piezometer at the point in question.

Absolute Pressure Line.—A line joining the absolute pressure heads for all points in the siphon barrel—also called the hydraulic grade line, a line joining the pressures as indicated by piezometers along the siphon barrel.

Energy Head.—The pressure head plus the velocity head at any point.

Energy Line.—A line joining the energy heads at each point in the siphon barrel.

Submergence.—The amount the summit section at its crown is drowned by water in the down-stream leg.

Let H = the siphon head as herein defined; m, the atmospheric head; E_{12}, the energy losses between Sections 1 and 2; E_{34}, losses between Sections 3 and 4, etc.; h, the velocity head; h_1 at Section (1), h_2 at Section (2), etc.; d, the difference in elevation of the inlet water surface and crown at the summit; and s, the depth the crown at the summit is submerged by water in the down-stream leg.

The energy losses in a siphon consist of those due to the entrance, conduit friction, and curvature. Each loss, and the total, may be expressed in terms of flow. The maximum velocity obtains at any point when the "absolute head" (as defined) is entirely consumed in supplying the preceding energy losses and the velocity head at that point. A Venturi section may be installed at any point within a siphon barrel and the entire static head plus one atmosphere, may be utilized on its throat, in producing velocity at, and providing for, the losses up to the throat. At such points of maximum velocity the "absolute pressure head" is zero.

It will be convenient to consider a siphon in two parts: (1) The up-stream leg from inlet to summit; and (2) the down-stream leg from the summit to the outlet. Fig. 10 shows such a siphon. The outlet is submerged and the down-stream leg is long enough to maintain a complete vacuum at the summit. (A "complete vacuum" is impossible, of course. The kind of vacuum that is meant is one that would exist under the following circumstances: Imagine a pipe, say, 36 ft long, with one end sealed, the other ending in a valve. Fill the pipe with water and close the valve. Place the pipe vertical with the valve submerged 1 ft in a tank of water; then open the valve. The water in the pipe will fall about 1 ft, leaving a foot of "complete vacuum" at the top of the pipe. The absolute pressure in the top of the pipe will be the vapor pressure plus some pressure from air released from the water.)

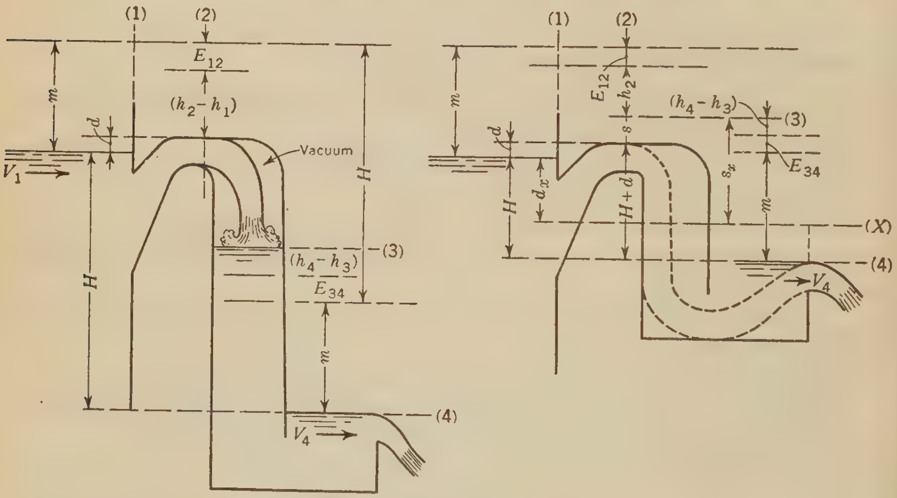

FIG. 10.

The down-stream leg is large in Fig. 10(*a*) so as to show the jet action of the water passing the summit. It is obvious that the velocity of the water will continue to increase under the action of gravity until it strikes the water at Section 3. If the velocity increases the transverse area must contract, so that the maximum velocity does not necessarily exist at the summit section. If the down-stream leg were shortened until Sections 3 and 2 coincided, the jet action would still exist, the velocity would increase, and the area would contract as long as the retarding force of water friction on the jet was less than the accelerating force of gravity. This is the phenomenon that was actually observed in the three Leaburg siphons. Figs. 7, 8, and 9 show that at Section 3 in each siphon practically the entire atmospheric head was consumed in overcoming losses and in producing velocity.

The condition shown in Fig. 10(*a*) will produce the maximum flow in a siphon. The entire absolute head on the summit is utilized in supplying the energy losses and producing the velocity through the summit section; thus,

$$m = d + (h_2 - h_1) + E_{12} \dots\dots\dots\dots\dots\dots (2)$$

That is, the atmospheric head must raise the water to the crown of the summit section, supply the energy losses, and increase the velocity from its initial to its final value. If the inlet is a fairly large body of water, as is generally the case, the velocity of approach, h_1, is zero and,

$$h_2 = m - d - E_{12} \dots\dots\dots\dots\dots\dots (3)$$

The atmospheric head on the outlet will be balanced by an equivalent column of water in the down-stream leg. This is true whether or not there is flow. If there is flow a head must be added sufficient to overcome the energy losses in the down-stream leg and also a head to produce the increment in velocity throughout that leg—that is, between Sections 3 and 4. The down-stream leg can be shortened until Section 3 coincides with Section 2, or it can be lengthened any amount without affecting the flow.

Next, imagine the down-stream leg shortened until Section 3 rises above Section 2. The flow is immediately decreased, and the condition is analogous to submerging an orifice. Fig. 10 shows such conditions. In the down-stream leg, Section 3 is now a distance, *s*, above the crown at the summit, which may be considered as the depth to which the summit section (orifice) is submerged. The summit section now discharges into something less than a vacuum and the flow correspondingly diminishes. In the down-stream leg,

$$s = m + E_{34} + (h_4 - h_3) - H - d \dots\dots\dots\dots\dots\dots (4)$$

The velocity head at the summit is then obtained from Equation (3) modified thus,

$$h_2 = m - d - E_{12} - s = H - E_{14} - (h_4 - h_3) \dots\dots\dots\dots (5)$$

The exit velocity from the up-stream leg now becomes the initial velocity for the down-stream leg, and $h_2 = h_3$. Hence Equation (5) reduces to,

$$h_4 = H - E_{14} \dots\dots\dots\dots\dots\dots (6)$$

That is, the velocity head at the siphon outlet is the siphon head less all energy losses. This is perfectly obvious.

The down-stream leg may be modified, as shown by the dotted lines in Fig. 10(*b*), without loss of generality. In the vertical down-stream leg the velocity will increase until it reaches a maximum with a corresponding contraction of the area of the jet. The point of maximum velocity will be where the accelerating and retarding forces balance. This point appears very difficult, if not impossible, of determination by mathematical analysis and is greatly complicated by the fact that the weight of water diminishes in proportion to its acceleration.

Referring again to Equations (5) and (6), it is noted that the velocity head at the summit section disappears; that is, the velocity in any section between inlet and outlet is indeterminate by such analysis.

The only manner in which the velocity at any point between inlet and outlet of the siphon barrel may be determined is by measuring it directly, by calculation from measured pressure head, or by assuming that the barrel is running full at that point, and dividing flow by area.

Data on the losses in curves of such conduits are very meager and unsatisfactory. The losses in Siphons Nos. 7 and 6 were calculated using available curve loss data,[4] while friction losses were computed by Kutter's formula with $n = 0.013$. The comparison between calculated and observed losses follow:

	Siphon No. 7	Siphon No. 6
Calculated:		
Entrance loss	1.0	0.6
Friction loss	9.0	3.2
Curve loss	14.6	11.6
Total	24.6	15.4
Loss by tests	17.2	21.1

The necessary assumption in the calculations that the barrels were running full was, of course, not an actuality, which may account for the anomalous results. The calculated losses for Siphon No. 7 were greater and those for Siphon No. 6 were less than the observed.

Coefficient of Flow

The generally accepted coefficient of flow is the ratio of the actual to theoretical flow. Where the transverse areas of the siphon barrels are not uniform, there has been some confusion as to which area should be used to determine the theoretical flow. This coefficient for the outlet area is written:

$$C_4 = \frac{Q}{a_4 \sqrt{2\,gH}} \qquad (7)$$

but, by Equation (6), $H = h_4 + E_{14}$ (notation as in Fig. 10), and,

$$h_4 = \frac{V^2_4}{2\,g} = \frac{Q^2}{a^2_4\, 2\,g} \qquad (8)$$

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[4] King's Handbook of Hydraulics, Second Edition, p. 193.

The energy loss, E_{14}, is the sum of all losses through the siphon, consisting essentially of entrance, friction, and bend losses. Each of these losses, and their sum, may be put in terms of flow having the form:

$$E_{14} = \frac{B\,Q^2}{2\,g} \quad \text{.............................} (9)$$

in which, B is a composite constant.

Substituting Equations (6), (8), and (9), in Equation (7),

$$C_4 = \frac{1}{\sqrt{1 + a^2_4\,B}} \quad \text{..........................} (10)$$

This flow coefficient is theoretically constant for a given siphon and is independent of the head on it or the flow through it (provided there is no appreciable amount of entrained air), since a and B are purely dimensional quantities. They can never be zero and, therefore, the coefficient for the outlet can never exceed unity. The factor, B, varies inversely as areas and directly as lengths. The coefficient, C, therefore, varies inversely as lengths and directly as all areas, except the outlet area for which it varies inversely.

The designer may diminish the coefficient by expanding the outlet, provided he can keep it full, at the same time increasing the flow and thereby increasing the efficiency of the siphon. Conversely, he may increase the coefficient by nozzling the outlet while he diminishes its flow and reduces the siphon's efficiency. If he keeps the outlet constant, he can increase the coefficient by increasing any or all areas above the outlet, or he can decrease it by lengthening the siphon barrel.

If the coefficient of flow be based on the summit area, the minimum area, or any other area, Equation (10) becomes,

$$C_x = \frac{a_4}{a_x}\,\frac{1}{\sqrt{1 + a^2_4\,B}} = \frac{a_4}{a_x}\,C_4 \quad \text{...................} (11)$$

If a_x is less than $C_4\,a_4$, then C_x becomes greater than unity. This can only happen, however, when the velocity due to the absolute head at any point is expressed in terms of the siphon head. For example, if a Venturi section with a throat area of 2.0 sq ft was put in Siphon No. 7 at Section 4, the flow would be reduced to 107.0 sec-ft, while the theoretical flow would be 85.4 sec-ft, resulting in a coefficient for the Venturi throat area of 1.26. The absolute pressure head at Section 4 that is used up in producing velocity through the throat, is 44.5 ft, of which, 21.1 ft is atmospheric. The siphon head is 28.4 ft. There appears to be no advantage in using other than the outlet area for computing the coefficient of flow. The writer suggests that the definition of the coefficient of flow for a siphon be adopted as the ratio of actual flow through the siphon to theoretical flow through its outlet area.

It is interesting to note that the coefficient of flow for any transverse section is also that for a unit area at that section; that is, it is the ratio of actual to theoretical velocity, since,

$$C_x = \frac{Q}{a_x\sqrt{2\,gH}} = \frac{V_x}{V_H} \quad \text{......................} (12)$$

It is obvious that a coefficient computed for a section that is not running full is meaningless.

If the water contains appreciable quantities of entrained air the coefficient will be reduced in proportion to the reduced flow. This is the most prevalent source of variable coefficients for the same siphon. If there is no entrained air the coefficient should remain constant for a given siphon regardless of variations in head and flow.

Efficiency of a Siphon

The efficiency of a siphon is not measured by its coefficient of flow because the designer can vary that coefficient by changing the dimensions of the siphon and can produce opposite variations in the efficiency. Consider, for a moment, the siphon as a machine. The efficiency as ordinarily conceived is the ratio of output energy to input energy. This is equivalent to the ratio of the outlet velocity head to the siphon head, that is (see Equation (12)),

$$\frac{h_4}{H} = \left[\frac{V_4}{V_H}\right]^2 = C^2_4 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(13)$$

Treated as a machine, therefore, the efficiency of a siphon would be merely the flow coefficient, squared. When referring to the efficiency of a pipe line, it is thought of as the ratio of the useful to the total head; that is, the ratio of $\frac{\text{Total head less losses}}{\text{Total head}}$, which is identical with Equation (13).

The purpose of a siphon is to discharge water through the utilization of atmospheric pressure. It would seem, therefore, that its efficiency is best measured by the ratio of the actual to its theoretically maximum flow. Such maximum flow obtains when one atmosphere is entirely used in producing velocity through the summit section, and up-stream energy losses are not included. This definition would lead to the expression (see Fig. 10(*a*)),

$$\text{Efficiency} = \frac{Q}{a_2\sqrt{2gm}} = \frac{V_2}{V_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots(14)$$

Siphons with low siphon heads would have low efficiencies and *vice versa.* A siphon with a long down-stream leg that contains a shear drop would have no greater efficiency than if the crown of the summit section were barely submerged. The efficiencies of the Leaburg siphons according to this criterion were, as follows:

	Siphon No. 7	Siphon No. 6	Siphon No. 5
Velocity at summit section from Table 3..	34.6	28.4	28.5
Velocity for one atmosphere............	46.1	46.1	46.1
Efficiency, percentage..................	75	61.5	61.6

The writer proposes to define the efficiency of a siphon as the ratio of the actual velocity through the summit section to the velocity corresponding to one atmosphere.

CONCLUSION

In conclusion, it may be observed that some of the phenomena and behavior of the siphons herein outlined could never be detected in the laboratory by the use of models. In such models, velocities never reach values high enough to disclose the jet action noticeable in the large siphon. The siphon barrel of a laboratory model will always run full after the siphon has been thoroughly primed. The barrel of the prototype may never run full, throughout its entire length. Nevertheless, much valuable information can be obtained by tests on models of siphons, once their limitations are fully recognized.

ACKNOWLEDGMENTS

In performing the tests described in this paper, the writer was assisted by Herbert H. Wheaton, Assoc. M. Am. Soc. C. E., and by the Operating Staff at the Leaburg Plant; the late Carl A. McClain, M. Am. Soc. C. E., Superintendent, also rendered valuable assistance and co-operated fully in the work.

DISCUSSION

IVAN M. NELIDOV,[5] ASSOC. M. AM. SOC. C. E. (by letter).—The outstanding feature of this paper is the demonstration of the fact that maximum vacuum occurs under certain conditions, not at the throat, as has been always assumed, but somewhat below the throat. The vacuum is also accompanied by a certain degree of contraction of the cross-section area. If the amount of contraction and the position of the contracted section could be expressed through sufficiently accurate coefficients, the conditions of flow and the efficiency of the siphon could be determined more accurately.

Before proceeding with the discussion of this paper the following constructive rather than critical comment is offered concerning the nomenclature, in order to simplify the terminology. "Summit" as introduced by the author has usually been termed "throat," and since, in most instances, it is a contracted section, it appears that the latter is more truly expressive. In accordance with the definitions of the Special Committee on Irrigation Hydraulics,[6] instead of "invert," it appears that the term, "crest," is more suitable. This is true especially because, during part of the operating time, the siphon functions as an overflow spillway. "Operating head" appears to be a more general definition than "siphon head," independent of the conditions of submergence at the outlet. The symbol, m, for atmospheric head is used by the Special Committee to denote mass. The writer proposes H_a for atmospheric head and H_o for operating head. The notations in this discussion are those of the main paper.

Referring to the theoretical analysis it can be mentioned that the theory of a siphon flowing full can be found in many books on hydraulics or irrigation.[7] In order to account for the flow through a siphon the influence of the centrifugal force and the contraction of cross-sections must be introduced. In theoretical investigations equations of uniform flow are used, in which all losses are taken proportional to the second power of velocity. Consequently, the coefficients expressing these losses should take account of the following factors: (1) Loss of head due to turbulent flow, loss of head due to partial filling, etc.; (2) local losses caused by curves, projections, entrance, outlet, etc.; and (3) uneven distribution of velocities across the section.

When a conduit begins to act as a siphon its efficiency as such may be reduced by partial filling and eddying. The necessity of designing it so that the barrel will be completely filled is apparent, and this condition will require smooth curves properly designed. On the other hand, a sharp curve is necessary, at least at the summit, in order to hasten and insure priming.

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[5] Senior Engr. of Hydr. Structure Design, State Dept. of Public Works, Sacramento, Calif.

[6] *Proceedings*, Am. Soc. C. E., May, 1932, p. 729.

[7] "Irrigation Practice and Engineering," by B. A. Etcheverry, M. Am. Soc. C. E., Vol. III, pp. 171–172, 1916.

The design requirements for a siphon may be enumerated as follows: (*a*) Priming time of long or short duration; (*b*) high or low priming head; (*c*) high or low rate of rise of forebay; (*d*) high or low coefficient of discharge; (*e*) breaking time of long or short duration; (*f*) protection against débris and ice; and (*g*) importance of eliminating vibrations and vortices. Combinations of these requirements may produce various types of siphons, best adapted to the conditions of the project.

Systematized knowledge necessary for satisfying these conditions is very meager. It is only known generally that: (*a*) Fast priming is produced by a perfect water-seal at the outlet, by sharp outlet curves, by a high rate of rise of the forebay, by a small height of the throat, and by small operating head; (*b*) low priming head is produced by good entrance conditions, by a water-seal that is not extremely high, by small outlet radius, and by small submergence of the discharge lip; and (*c*) high efficiency is produced by smooth curves, Venturi-shaped siphon, and certain height of water-seal in respect to the operating head.

Most of the existing siphons represent complex hydraulic structures. They have bell-shaped entrances of irregular form, widely varying cross-sections, projections, positive and negative inclinations of the outlet leg, various shapes of outlet, etc. Experiments on hand do not supply complete information for the designer. He is forced to use the same old "Weisbach formula" for losses in curves and to guess other numerical factors. Available data are obtained usually on small scale structures, in which the influence of the force of gravity could not properly manifest itself. There is now an urgent need for quantitative data which could be used in design with a fair degree of certainty. The experiments should supply data not only for siphons running full, but also with various quantities of air carried by flow.[8]

Under present conditions the analytical study of the performance of a siphon is applicable only to the evaluation of discharge. An analytical interpretation of factors other than discharge, contributing to the action of a siphon, such as priming head, priming time, etc., is impossible, because of the many variables involved, or due to the influence of the unstable conditions of flow. Empirical formulas will probably approach the solution in the form of exponential functions.

The definition of efficiency of a siphon by the author, as expressed in Equation (13), agrees with that of Professor J. N. LeConte.[9] As the author correctly states engineers are interested in discharge efficiency. In the writer's opinion, it will be as follows:

For $H > h_m$ + outlet losses + difference of the outlet and summit velocity head:

$$e = \frac{Q \text{ (actual)}}{Q \text{ (theoretical)}} = \frac{C_o \sqrt{2 g m}\, a_o}{\sqrt{2 g m}\, a_n} = C_s \frac{a_s}{a_n}$$

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[8] "Experimentation upon Model Siphons Sucking Air," Thesis by Messrs. William Saylor and David W. Anderson, Univ. of California, Berkeley, Calif.

[9] "Hydraulics," by J. N. LeConte, p. 48, 1926.

For $H < h_m$ + outlet losses + difference of the outlet and summit velocity head:

$$e = \frac{Q \text{ (actual)}}{Q \text{ (theoretical)}} = \frac{C_0 \sqrt{2\,g H}\, a_0}{\sqrt{2\,g H}\, a_n}$$

in which, a_n is the narrowest area of cross-section of the siphon.

The difference in the elevation of the summit and the reservoir water level is neglected in these definitions.

HERBERT H. WHEATON,[10] ASSOC. M. AM. SOC. C. E. (by letter).—In the "Conclusion" to his paper Mr. Stevens observes that "some of the phenomena and behavior of the siphons herein outlined could never be detected in the laboratory by the use of models." The writer feels that he should add a few remarks about the tests he made of models of Siphon No. 7 (see Fig. 3(*c*)) of the Leaburg siphons, at the hydraulic laboratory at the University of California, shortly after he assisted Mr. Stevens in making the field tests.

Models of Siphon No. 7 to scales of 1: 20, 1: 15, and 1: 7.5 were tested. The 1: 20 model was fitted with piezometer tubes corresponding to those in the prototype. The pressure readings at Sections 1 and 2, intake and throat (see Fig. 7), checked those in the full scale almost exactly. At the other sections this was not the case, because the model ran full.

The coefficients of discharge of the models, based upon the formula, $Q = CA \sqrt{2gH}$, were 0.68 for the 1: 20; 0.69 for the 1: 15; and 0.71 for the 1: 7.5 scales. The coefficient corresponding to these for the original was 0.63. Had the full-scale siphon run full it would have shown a coefficient of discharge of something in excess of 0.71, the value of which could be found by extrapolation from the model results. The model tests demonstrate the maximum coefficient for a siphon of this design. The scale effect is due to the fact that the discharge formula is based upon the assumption that only gravitational forces influence the discharge. The coefficients given for the models are constant for velocities in the models which give a value of Reynolds' number greater than some fixed value which is the same for all scales of model[11].

The writer thought that Siphon No. 7 of the Leaburg siphons probably leaked enough air around the movable air-vent cover-plate to have some effect upon the discharge. He made tests, therefore, to determine the effect of air leakage upon the 1: 20 and 1: 15 scale models. He found that the coefficient of discharge for each of these models could be reduced to 0.63 if the model were allowed to suck from 3 to 4½% of air at the air vent; that is, if the volume of air measured at atmospheric pressure were 3 to 4½% of the volume of water passing through the siphon. Although no doubt some air leakage occurred at the intake of the prototype, the writer is not prepared to say whether it could have been in such quantities.

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[10] Fresno, Calif.

[11] "Experiments on Siphon Spillways," by A. H. Gibson, T. R. Aspey, and F. Tattersall, *Minutes of Proceedings*, Inst. C. E., Vol. 231, 1931.

Mr. Stevens mentions that Siphon No. 7 primed with a head of water over the crest of 0.2 to 0.3 ft. The three models, when air-tight and sucking no air at the intake, checked each other almost exactly as to the head necessary to prime them for corresponding rates of rise of the forebay water level, but all showed priming heads far in excess, correspondingly, of those in the original. When the water rose in the forebay it could not rise to the same level inside the siphon throat because of air compression within the tube. The quantity of water flowing over the crest was not enough to produce a velocity in the sealing basin sufficient to carry the air out to the outlet until a much higher level, correspondingly, than 0.3 ft above the crest was reached in the forebay.

The writer concluded that the air pressure within the tube of the prototype never became greater than atmospheric, or at the most very little greater, perhaps because some air could escape around the air vent, and also because of the larger volume of the tube. He, therefore, connected the throat of the 1 : 7.5 model with the outside air by a $\frac{3}{16}$-in. rubber hose so that compressed air could escape during priming. Observation of the water levels both in the forebay and within the siphon showed that they remained the same. The model then primed at heads that corresponded almost exactly with a head of 0.3 ft in the full scale. It was interesting to note that the tube could be left in the throat all during the priming, permitting air to enter the throat when pressure there became less than atmospheric, and that this made no noticeable difference in the head necessary for priming. After the velocity in the sealing basin became sufficient to carry the air bubbles around the lower bend, the tube was evacuated so rapidly that the small hose opening made no appreciable difference.

J. C. STEVENS[12], M. AM. SOC. C. E. (by letter).—The writer's proposal that the coefficient of flow of a siphon be defined as the "ratio of actual flow through the siphon to theoretical flow through its outlet area," has not been challenged. It should be accepted, therefore, by the profession.

The writer suggested further that the discharging efficiency of a siphon be defined as the "ratio of the actual to its theoretically maximum flow," and that the theoretically "maximum flow obtains when one atmosphere is entirely used in producing velocity through the summit section" (neglecting up-stream losses).

This is expressed by Equation (14) and may be easily remembered as the ratio of the velocity through the summit section to the velocity corresponding to one atmosphere. Mr. Nelidov offered a substitute which, if the writer understands correctly, states that the efficiency is equal to the coefficient of flow for the summit section. As the writer pointed out, this is illogical because, if the summit section is constricted, an efficiency in excess of unity is possible. The writer's proposed definition should be accepted.

Mr. Nelidov's criticism of terms deserves mention. "Throat" is not synonymous with "summit." The summit (highest) section of the siphon is meant; not the smallest section. The term, "siphon head," was used to mean the difference in water-surface elevations of head and tail-water. It is not always equivalent to "operating head" which frequently refers to forebay level only. As defined by Professor Etcheverry[7] the "operating head" is measured to the center of the outlet, while "siphon head," as used by the writer, is measured to the water surface at the outlet. "Crest" might have been used instead of "invert," but a distinction was necessary between the upper and lower boundaries of the summit section, and "crown" and "invert" are common terms for these boundaries.

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[12] Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

The writer tried to interest laboratory experimenters in determining losses by laying the model siphon barrel horizontally with its inlet and outlet connected to a head and tail-water tank, respectively. The first to test models of existing siphons in such a manner will learn something about siphon losses. Heretofore, these losses have been considered simply those of inlet, bend, and friction, but the tests cited show that cavitation and, therefore, impact losses must also be taken into consideration. Such impact losses are not subject to analysis by the ordinary model studies, because the velocities to produce them can not be secured. That is what the writer meant by the statement quoted by Mr. Wheaton.

By placing the model siphon horizontally, however, between a pair of tanks where a range of velocities from those in the model to those in the prototype can be secured, cavitation and impact losses can be isolated from those of bends, etc. Professor, E. W. Spannake[13] has been doing some interesting work on cavitation at the laboratory of the Massachusetts Institute of Technology.

Mr. Wheaton attributes the low discharge coefficient, 0.63, of Siphon No. 7 to entrained air, and attempts to show that from his model tests the coefficient should be 0.71 or more if the siphon runs full. As designed, the siphon can never run full. Moreover, there are losses from cavitation and impact, which the writer believes are the real reasons for the differences in the coefficients between the model and the prototype. In the case of Siphons Nos. 5 and 6, entrained air is conceded to have affected the results, giving discharge coefficients of about 0.48, entirely too low; but cavitation also existed.

In Mr. Wheaton's models, air was compressed in their summits as the forebay level, increased, which prevented them from priming except at heads much greater than those corresponding to the prototype. He suggests that the priming hood of the prototype leaked enough to prevent air from being compressed in its summit, but not enough to delay priming. This reasoning does not accord with the facts as later ascertained.

At the writer's suggestion the chief operator of the Leaburg Plant made some experiments to determine the behavior of the siphons as regards priming with the hoods as built with some leakage and also when sealed against air leakage. The lower edges of the priming hoods were not in their lowest position, as was the case when the tests given by the writer were made; hence, the heads necessary to prime are greater than in the writer's tests, but the relative values for sealed and unsealed hoods are very illuminating. The priming gate on Siphon No. 7 was 0.11 ft and that of Siphon No. 6 was 0.19 ft above the invert of crest of the summit section.

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[13] Current Hydraulic Research: Repts. Nos. I-3 and II-1, U. S. Bureau of Standards (Ref. No. VI-6 I N H-655-C).

For the sealed condition, the priming gate (see "Behavior in Operation" and Fig. 3) was held tightly against its lower guide by iron wedges, and sealing was effected with heavy gear grease. Following are shown the heads (difference, in feet, between siphon crest and forebay water level) to prime:

	As built unsealed	Sealed
Siphon No. 7	0.52	0.30
Siphon No. 6	0.53	0.39

If these priming heads are corrected for the distances of the priming gates above the siphon crest (since that depth of water flowed over the crest before the air was shut off) the actual priming heads are found to be, in feet:

	Unsealed	Sealed
Siphon No. 7	0.41	0.19
Siphon No. 6	0.34	0.20

Each of the foregoing tests represents an average of three trials. They show that the sealed siphon primed, roughly, at one-half the head of the unsealed siphon, showing that the air was expelled much faster than it could be compressed by the rising forebay. The leakage around the priming gate increases the time and head of priming instead of reducing it, as the models seemed to indicate. In this regard, the model failed completely to predict the actual behavior of the siphon, and points to another limitation that should be recognized in model work.

Interesting data were obtained during these tests. In order to prevent Siphon No. 7 from priming while Siphon No. 6 was under test, it was necessary to insert air pipes under the hood of No. 7, having an aggregate area of 12.5 sq in., or 2.5% of the area of the smallest section of the siphon barrel.

Mr. Nelidov mentions ice. In 1932 a sharp, cold period brought down some frazil ice and also formed surface ice in the air inlet to the siphons, enough to choke them. The barrels were not affected, but the primers were so choked that the siphonic action would not cease until the water had been drawn below the upper edge of the inlets (see Fig. 3), or 5.25 ft below normal operating level. In order to avoid a repetition of this trouble, holes have been drilled through the concrete to the crowns of the summit sections and 6-in. iron pipe nipples have been cemented into them. These nipples are closed by a screw cap which can be removed to let in air to break the siphonic action whenever the priming hoods become choked with ice.

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USE AND CAPACITY OF CITY STREETS

By Hawley S. Simpson,[1] Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. R. O. Thomas, Sigvald Johannesson, W. W. Crosby, Harold M. Lewis, and Hawley S. Simpson.

Synopsis

City passenger transportation is provided by two distinct classes of vehicles; (*a*) Individual units, such as passenger automobiles and taxicabs; and (*b*) mass transportation vehicles, including street cars, motor buses, and (to a small extent) trolley buses. Subways and elevated lines are omitted from Classification (*b*) because they operate on private rights of way. The passenger-carrying ability, or passenger capacity, of vehicles of these classes varies within wide limits, and thus the utility of the street system, bears a direct relation to the type of unit used for passenger transportation.

In this paper particular attention is directed to the passenger capacity of city streets when used solely by passenger automobiles and when used jointly by private and public vehicles. Special consideration is given to the effect that parking has in reducing both vehicle and passenger capacity. Detailed tables are included showing vehicle and passenger capacities of streets of different widths when parking is allowed and when it is prohibited. These tables have been prepared after a study of a large number of theoretical analyses and field surveys of actual street conditions and capacities, and are believed to provide a reasonably reliable measuring stick of potential street capacities. It will be found, in many cases, that street usage on thoroughfares commonly considered to be "congested," falls far short of the limits here developed, for congestion more often occurs because of deficient traffic control and regulation than because of traffic saturation.

Improvement in city traffic conditions can result only when careful attention is given to the relative economic importance and efficiency of both private

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Note.—Published in August, 1933, *Proceedings*.

[1] Research Engr., Am. Transit Assoc., New York, N. Y.

and public vehicles. Street improvement projects, as well as traffic regulation plans, should be developed so as to provide the highest possible facility of movement for both private and public vehicles. In this way the services rendered by each may be developed to fill the public need for transportation most satisfactorily.

This study naturally shows very much greater passenger capacities on streets served by mass transportation vehicles, despite the fact that the total vehicle capacity suffers a reduction. This merits the serious consideration of public authorities faced with the necessity of augmenting street facilities.

INTRODUCTION

In the modern scheme of city life the transportation problem takes foremost rank. None is more difficult to treat to the satisfaction of all, because none, perhaps, so closely touches every phase of social, commercial, and industrial activity.

The basic pattern of local transportation is built around the so-called "necessity" travel—from home to work in the morning and again from work to home in the evening. To this are added the incidental movements of traffic upon which the efficient functioning of every metropolitan area depends. All these matters are in a continual state of flux. Habits and places of living, work, and play are constantly changing. Decentralization of homes and of commercial and industrial establishments has been taking place for some time. Simultaneously, in some places, there has been a noticeable tendency toward recentralization; many individuals now find it more convenient to live near the center of activity, and certain types of businesses are recognizing new advantages in crowded areas. Social and recreational demands of the community produce their own transportation problems. In this respect, too, change is evident. Simple home, or neighborhood, amusements no longer suffice. Young and old alike seek outside entertainment more frequently and at greater distances from the home, while travel, as such, is becoming an important means of recreation.

If the transportation system of a city—embracing streets and public and private facilities for travel—is properly conceived the nature and direction of city growth may be moulded in ways calculated to promote the best interests of the community. Improperly organized, growth may be stifled or may take place when and where the streets are inadequate for all essential requirements, with corresponding economic loss. Knowledge of the potential ability of city thoroughfares to serve the public needs is essential, therefore, to all who are in any way concerned with the administration of municipal affairs.

This paper has been prepared in the course of studies that necessitated the collection of many widely scattered data on the subject of street use and street capacity. It has not been possible to conduct the extensive research necessary to lift the subject from the realm of controversy; almost complete reliance has had to be placed upon existing surveys. The insufficiency of this information will be readily apparent to the reader and, it is hoped, may serve to encourage further study of the problems involved in moving passengers and merchandise through city streets with particular attention to the capacity characteristics of public ways.

Street capacity is usually measured in numbers of vehicles passing a given point in some unit of time—usually 1 hr. Vehicles, however, are only the instruments of transportation, the primary function of a street system being to provide for the movement of persons and merchandise and to give public access to abutting private property which thus gains value by becoming accessible to those who wish to live, work, shop, play, or engage in commerce or industry thereon. Land, in itself, has little inherent worth. Richard M. Hurd has stated:[3]

> "In cities, economic rent is based on superiority of location only. * * * thus the value of all urban land ranges from that which least serves the smallest number of people of the lowest economic quality, up to that which best serves the largest number of people of the highest economic quality."

On this account the passenger capacity of a city street is of much greater social importance than its vehicle capacity. Therefore, in this paper, attention is directed primarily to the passenger capacity of streets, beginning with an analysis of their vehicle capacities.

The Meaning of Street "Capacity"

"Capacity" is a difficult word to define when used with reference to a street system. It conveys an implication that there is some definite limit to the number of vehicles that may move along a street in a stated period of time. No such rigid interpretation is possible. Were the carrying ability of a street to be taxed to its ultimate—in other words, were its theoretical maximum capacity to be reached—conditions would result which would not long be tolerated. Assuming that such a condition should occur on some one street, motorists would be inclined to divert to near-by, less direct, and more lightly traveled highways until a condition of stability was reached, with the more favorably located streets somewhat more densely used than the others. Under such conditions it might be said that the streets were carrying capacity loads; yet each might be capable of accommodating considerable additional traffic if it were to present itself and if no convenient alternative streets were available.

Obviously, then, the capacity of a street is not capable of accurate measurement. For present purposes, it will be sufficient to state the sense in which the word, "capacity," is to be used:

> "The capacity of a city street is reached when the volume of traffic is so great that any further increase will result in conditions of movement so unsatisfactory to the users that less favorably situated routes are preferred by a considerable proportion of those who would normally find the original street most useful."

It is obvious, from this statement, that the "capacity" of any particular street, as the word is here used, will be largely dependent upon the relative utility of, and volume of traffic upon, near-by streets. Thus, the traffic capacity of the "Fifth Avenues" of large cities will be considerably higher, generally, than that of the "Main Streets" of smaller cities. By confining investigations to conditions as they exist in very large cities, the results may be considered as most applicable to streets that are commonly considered "congested," and indicative, therefore, of reasonable limiting conditions consistent with satisfactory speed and comfort of travel.

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[3] "Principles of City Land Values," by Richard M. Hurd, 1905, pp. 1 and 13.

VEHICLE CAPACITY OF HIGHWAYS WITHOUT GRADE CROSSINGS

The number of vehicles per lane which will pass a given point in unit time is a function of their average speed and center-to-center spacing, as expressed in the equation:

$$N = 5\,280 \left(\frac{V}{P}\right) \qquad (1)$$

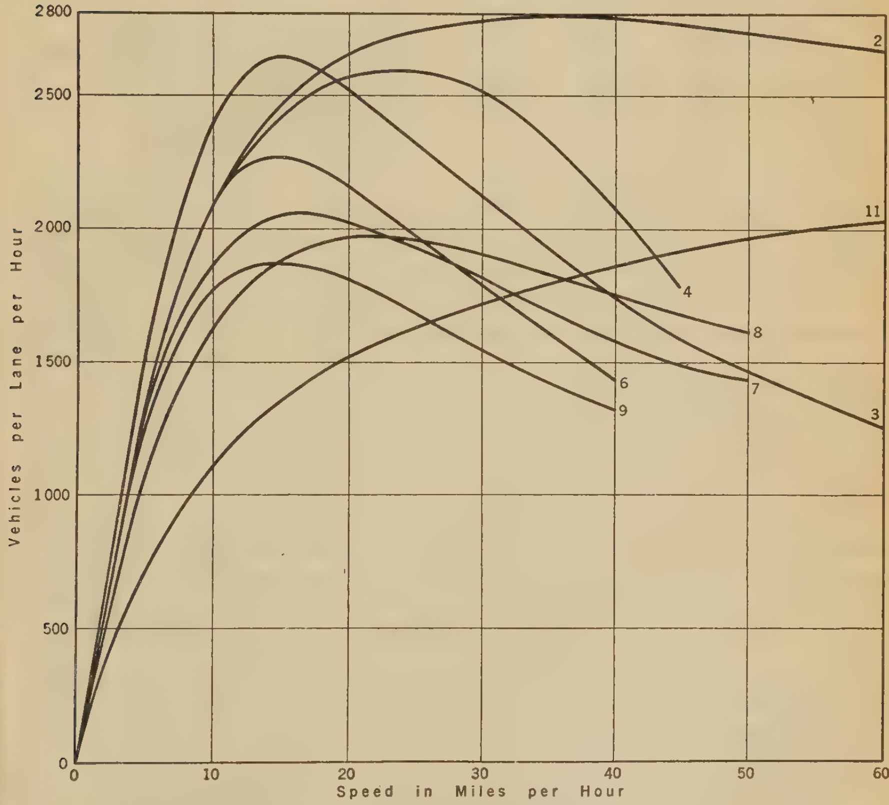

FIG. 1.—VARIOUS DETERMINATIONS OF THE HOURLY NON-STOP CAPACITY OF A SINGLE LANE OF VEHICLES (SEE TEXT FOR SOURCES OF DATA).

in which, N is the number of vehicles per lane per hour; V, the velocity of vehicles, in miles per hour; and P, the average center-to-center spacing of vehicles, in feet.

Many attempts have been made to determine, through theoretical studies and by actual observation, the capacity of a single lane of vehicles uninterrupted by cross-traffic. Fig. 1 presents several of the curves, and Table 1 shows the computed maximum capacity and the speed at which each is estimated to occur. Maxima determined by others, and not recorded on Fig. 1, are also included as Item Nos. 1, 5, and 10, in Table 1.

TABLE 1.—Various Determinations of the Hourly Non-Stop Capacity of a Single Lane of Vehicles.

Item No. (see Fig. 1)	Capacity (maximum vehicles per hour)	Speeds, in miles per hour	Spacing, from center to center, in feet	Conditions
1.....	3 320	20.8	33	Minimum spacing of individual vehicles.
2.....	2 800	34.5	65	....
3.....	2 640	15.	30	....
4.....	2 600	23.5	48	....
5.....	2 430	20.7	45	Average group spacings.
6.....	2 264	15	35	"Emergency stop" spacing, 50% four-wheel brakes and 50% two-wheel brakes.
7.....	2 045	16.5	43	Four-wheel brakes on all vehicles.
8.....	1 969	22	59	"Ideal" conditions.
9.....	1 880	15	42	Average group spacings.
10.....	1 852	24.9	71	Average spacing of all vehicles.
11.....	.....	....	..	No maximum reached (see Fig. 1).

The information contained in Table 1 was obtained from various sources, as follows: Items Nos. 1, 5, and 10 were reported by R. F. Kelker, Jr.,[3] M. Am. Soc. C. E.; Items Nos. 2 and 3, by A. N. Johnson,[4] M. Am. Soc. C. E.; Item No. 4, by the Regional Planning Commission, County of Los Angeles, California,[5]; Items No. 6,[6] No. 7,[7] and No. 8,[8] by the Committee on Highway Traffic Analysis, Highway Research Board; Item No. 9, by Harold M. Lewis and E. P. Goodrich,[9] Members, Am. Soc. C. E.; and Item No. 11, by Sigvald Johannesson,[10] M. Am. Soc. C. E. The data for Curve 11, Fig. 1, are not given in Table 1 (see Item No. 11), because no maximum is reached regardless of speed.

The wide variation in the results shown in Fig. 1 seems to confirm the statement that maximum street capacity can not be accurately ascertained. Nevertheless, surveys in several places show rather comparable results of actual performance, indicating that when the traffic volume has reached a point at which conditions are generally unsatisfactory, a proportion of the drivers seek other thoroughfares. The results of representative surveys appear in Table 2, for which the following authorities may be recorded: Item No. 1, by A. N. Johnson,[11] M. Am. Soc. C. E.; Items Nos. 2 and 3, by the American

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[3] *Journal*, Western Soc. of Engrs., April 19, 1926, pp. 140–147.

[4] *Proceedings*, Eighth Annual Meeting, Highway Research Board, 1929, p. 115.

[5] *Proceedings*, Thirty-sixth Annual Convention, Am. Soc. of Municipal Engrs., 1931, p. 47.

[6] *Proceedings*, Seventh Annual Meeting, Highway Research Board, 1928, p. 233.

[7] *Proceedings*, Ninth Annual Meeting, Highway Research Board, 1930, p. 106.

[8] *Proceedings*, Seventh Annual Meeting, Highway Research Board, 1928, p. 231.

[9] Regional Plan of New York and Its Environs, "Highway Traffic," 1927, p. 86.

[10] "Highway Economics," McGraw-Hill Co., 1931, p. 91.

[11] *Proceedings*, Eighth Annual Meeting, Highway Research Board, 1929, p. 108.

Transit Association[12]; Item No. 4, by the Highway Research Board, Committee on Highway Traffic Analysis[13]; Item No. 5, by the Cleveland Railway Company[14]; Item No. 6, by the Detroit, Mich., Rapid Transit Commission[15]; Item No. 7, by Ole Singstad, M. Am. Soc. C. E.; Item No. 8, by E. A. Byrne,[16] M. Am. Soc. C. E.; and, Item No. 9, by Day and Zimmerman, Inc.[17] Referring to Item No. 7, the capacity of the Holland Tunnel is computed by the Engineers of the Port of New York Authority as 1 270 vehicles per hr per lane, in two lanes, when most of the traffic is composed of passenger vehicles. M. O. Eldridge, Assoc. M. Am. Soc. C. E., has stated[18] that delays incident to the purchase and collection of tickets, the weaving of vehicles from several lanes into two at the tunnel entrances, and the grades within the tunnel limit the capacity of the tubes.

TABLE 2.—MAXIMUM OBSERVED RATES OF TRAFFIC FLOW PER AVERAGE LANE—ONE DIRECTION.

Item No. (1)	Location (2)	Number of lanes in each direction (3)	MAXIMUM HOURLY RATES						Estimated capacity per lane per hour † (10)
			20 sec. (4)	1 min. (5)	5 min. (6)	10 min. (7)	30 min. (8)	60 min. (9)	
1	Baltimore-Washington Highway	1	.....	.....	1 968	.....	.....	1 502	1 500
	Lake Shore Drive, Chicago, Ill.:								
2	Average for both lanes	2	2 070	1 830	.....	1 392	1 365	1 349	1 335
3	Inner lane	2	2 520	2 160	.....	1 542	1 412	1 395	.....
	Superior-Detroit High-Level Bridge, Cleveland, Ohio:								
4	1928	3	.....	.....	.....	.....	.....	1 241	1 170
5	1927	2½*	.....	.....	1 958	1 874	1 699	1 557	1 230
6	East Grand Boulevard, Detroit, Mich.	1	.....	1 800	.....	1 404	1 200	.....	.....
	New York, N. Y.:								
7	Holland Tunnel	2	.....	.....	.....	.....	.....	1 253	1 335
8	Queensborough Bridge	3	.....	.....	.....	.....	.....	1 482	1 170
9	Manhattan Bridge	2	.....	.....	.....	.....	.....	1 300	1 335

* Estimated number of effective lanes.

† Calculated by using the percentages of Table 3, multiplied by 1 500 (the estimated hourly non-stop capacity of a single lane).

In none of these surveys can it be stated definitely that non-interrupted traffic was observed or that sufficient traffic existed to fill the street completely. Photographs taken during the heaviest 20-sec flow on the Lake Shore Drive, in Chicago, Ill., showed material gaps in both lanes which could have accommodated other vehicles (if the supply had existed) at the rate of about one for every three observed. (See Table 2, Item No. 2.) If this possible density of use had occurred, the rate of flow would have been about 2 800 vehicles per lane per hr, being only about 500 vehicles per hr less than found in the observations by Mr. Kelker, based upon minimum spacings of individual vehicles, and checking exactly with the figure developed by Dean Johnson. (See Table 1, Items Nos. 1 and 2.)

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[12] Rept. of Comm. on Street Traffic Economics, Am. Elec. Ry. Assoc. (now Am. Transit Assoc.), 1928, p. 69.

[13] *Proceedings*, Eighth Annual Meeting Highway Research Board, 1929, p. 92.

[14] Survey Rept. on Superior-Detroit High-Level Bridge, June 18, 1927.

[15] Rept. on Vehicular Traffic in the Business District of Detroit, 1924, p. 16.

[16] *Civil Engineering*, July, 1931, pp. 934-935.

[17] Rept. on Highway Traffic Conditions and Proposed Traffic Relief Measures for the City of New York, 1929, p. 163.

[18] Rept. on Traffic Capacity of Holland Tunnel, 1931 Convention, Am. Road Builders Assoc.

Perhaps the ultimate capacity of a lane, for very short periods, may approach 3 600 vehicles per hr. At this rate, and at speeds of 22 miles per hr, observed in the Chicago case (Items Nos. 2 and 3, Table 2), the center-to-center spacing of vehicles would be about 32 ft. This does not seem unreasonable as the limiting figure. No such capacity figure should be used for design purposes, however. Maximum short-period traffic flows, as seen from Table 2, are almost twice as high as the hourly flow. A rate of flow for short periods as high as the probable ultimate will occur when the hourly traffic is about 1 800 vehicles per lane. The acceptance of such a figure for working purposes would leave only a small factor of safety, however. For present purposes a value of 1 500 vehicles per single lane per hr may be adopted as representing "capacity," with the realization that higher flows are possible and in the belief that the factor of safety allowed is no greater than is necessary to maintain reasonably fluid movement at speeds consistent with safety and efficiency.

Multiple-Lane Highways Without Grade Crossings

The value, 1 500 vehicles per single lane per hr, for lane capacity is predicated upon a single moving lane in each direction. On wider streets vehicles seldom travel in well-defined paths, but weave from lane to lane and straddle the imaginary line between lanes, thus occupying space potentially capable of accommodating two or more vehicles and reducing the street capacity. In order to overcome this condition, rigid restriction of each vehicle to its original lane is being practiced in a few instances. In the Holland Tunnel, between New York City and Jersey City, N. J., police patrols effectively prevent motorists from crossing the painted strip separating the two lines in each one-way tube. On the new upper level of the Queensborough Bridge, in New York City, an attempt was made to control the alignment of automobiles by constructing tracks for three lines of traffic in one direction.[16] This resulted in lanes which seemed unsafe and at the protest of motorists the tracks were removed so that the roadway now (1933) handles only two lanes.

TABLE 3.—Relative Capacity of Multiple-Lane Streets, Assuming the Single Lane as 100 Per Cent

Number of lanes in one direction	Efficiency per lane, percentage	Capacity of half street, percentage
1	100*	100
2	89†	178
3	78†	234
4	65‡	260
5	52‡	260
.....	...	...

*Assumed. † Actual.

The effect of weaving and straddling on non-stop highways has been investigated by Mr. Kelker.[8] The results are shown in Table 3. From this it appears that an express highway with more than three moving lanes in each direction (a total of six lanes) is of doubtful economy. The construction of the fourth lane, adding 33⅓% to the width, increases the capacity of the street only about 11%, while a street with five lanes in each direction has no greater capacity than one of four lanes.

Using the data of Table 3 and the estimated capacity of a single lane (1 500 vehicles per hr), the capacities of grade-separated highways of from one to three lanes have been computed (Table 2, Column (10)) and compared with the observed maxima of Table 2, Column (9). In studying this comparison it should be borne in mind that the estimated capacities have been adopted with a view to allowing a reasonable factor of safety, and it is to be expected that at times they will be exceeded.

VEHICLE CAPACITY OF SURFACE STREETS

Uninterrupted traffic flow is possible on only a few of the present highways. Surface street traffic is subject to frequent interruption by crossing vehicles or by "stop-and go" signals. The capacity of any single street could be approximated if the timing and spacing of signals, or the volume of cross-traffic, or both, were known, and if speed of vehicles and other pertinent factors were available; but such computations are of practical value only in specific instances. For the purpose of this paper it will be assumed that the capacity of a traffic lane devoted to straight-through movement on a surface street is equal to 60% of its non-stop capacity, namely, 900 vehicles per lane per hr. This applies, of course, only to major streets on which the traffic volume is approximately twice as great as that on the cross-streets. Traffic volumes exceeding 900 vehicles per lane per hr have been observed on streets with traffic signals and with grade crossings; but such streets are not the average that must be dealt with in general observations.

FACTORS AFFECTING CAPACITY OF MULTIPLE-LANE SURFACE STREETS

The capacity of multiple-lane surface streets cannot be determined in the same manner as in the case of grade-separated highways. Weaving and straddling doubtless reduce street capacities somewhat, but they are probably not very important except on unusually wide surface highways as, for example, streets of more than eight moving lanes. Other factors, not common to non-stop highways, affect surface street capacities so greatly that the relative importance of weaving and straddling is almost negligible on streets of average width. Some of the major factors that require consideration are: Turning movements; parking; street car traffic; and bus traffic.

Turning Movements.—Left-hand and right-hand turns reduce the capacity of the lanes from which each is made by delaying the traffic (behind the turning vehicle) during the maneuver. To determine accurately the effect of left-hand turns upon capacities, the traffic flow and incidence at each intersection should be known for traffic in the opposite direction as well as for the traffic that turns. Pedestrian volumes and direction would also be essential.

In the case of right-hand turns, the volume of turning traffic and the pedestrian traffic in the original direction of the turning vehicle should be known. There being no recorded observations of such character it has been assumed that the delay to traffic in turning lanes will average 15% of its moving time by left turns and 5% by right turns. If a single lane must accommodate through traffic as well as both right and left turns, the reduction in capacity would be the sum of these percentages.

Parking.—Curb parking eliminates one entire traffic lane and probably somewhat lowers the capacity of the lane nearest thereto because of double parking and conflict between vehicles moving to and from the curb lane. The exact measure of this factor would depend upon the prevalence of double parking and the frequency of parking line interchange. For present purposes it will be sufficient to assume, in general, that the capacity of the lane next to the parked lane is reduced 5% under ordinary regulations permitting parking.

If parking is not allowed the curb lane nominally becomes available for moving traffic, but since stopping for the discharge of passengers and merchandise is usually permitted, the capacity of the curb lane for moving traffic may vary between wide limits, depending upon the extent of the desire or need to stop. Observations in Chicago have shown traffic flows in the curb lane on central district streets exceeding a rate of 600 vehicles per hr during the maximum half-hour, when parking is prohibited.[19] The results of similar observations in Detroit, Mich., are shown in Table 4.[20] Referring to Item No. 4, the third or center lane was used by 51 street cars and 634 automobiles. On Jefferson Avenue (Item No. 5, Table 4) the fourth or center lane was used by 41 street cars, 25 buses, and 25 automobiles. Therefore, in these two items, the last, or center, lanes were not included in computing values in Column (5) of the table.

TABLE 4.—USAGE OF CURB LANE DURING MAXIMUM HOUR, IN DETROIT MICHIGAN (PARKING PROHIBITED).

Item No. (1)	Location (2)	Number of one-way free lanes (3)	Vehicles per hour in curb lane, parking prohibited (4)	Curb-lane vehicles as percentage of average vehicles per lane in adjacent lanes (5)
1	John R. Street, at East Grand Boulevard......	2	384	70
2	West Grand Boulevard, at 14th Street........	3	295	40
3	West Grand Boulevard, at Cass Avenue......	4	543	106
4	Grand River Avenue, at West Grand Boulevard	3	830	93
5	Jefferson Avenue, at East Grand Boulevard...	4	707	78

On the basis of these observations it seems conservative to assume that the average curb lane, when parking is prohibited, and allowing for the influence of right turns, is the equivalent of at least one-half a free lane—450 vehicles per hr. If left turns must be made from this lane its effectiveness will be reduced 15% as previously stated.

Effect of Street Cars on Automobile Capacity.—Obviously, there must be some relation between the motor-vehicle capacity of a street-car lane and the number of street cars operated on it. If the tracks are used by only a small number of street cars, motor vehicles should be able to use that lane with practically the same freedom as the adjoining free lane. If, on the other hand, street-car headways are very short, so that the maximum capacity of the track in street cars per hour is reached or approached, efficient operation of street cars implies a minimum of motor vehicles in the street-car lane. Such conditions may hold, however, for only a short time in the rush period while during the remainder of the day motor-vehicle operation on the car tracks may not greatly hinder street cars. Data pertaining to this subject have been secured from Detroit[21] and Chicago[22] and are given in Table 5.

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[19] Rept., Committee on Street Traffic Economics, Am. Elec. Ry. Assoc. (now Am. Transit Assoc.), 1930, p. 59.

[20] "Vehicular Traffic in 1930", Detroit Rapid Transit Comm., 1930, p. 75.

TABLE 5.—OBSERVED HOURLY RATE OF TRAFFIC FLOW IN STREET CAR AND ADJACENT FREE LANES*

City and street	Period, in minutes	In Street-Car Lane per Hour		Automobiles per lane in adjacent free lanes per hour
		Street cars	Automobiles	
Chicago, Ill.:				
Broadway	10	60	568	270§
Broadway	30	34	506	212§
Broadway	60	34	432	205§
Broadway	60	23	471	281§
Broadway	60	36	265	323§
Broadway	60	30	618	567§
Broadway	60	30	275	299§
Broadway	60	27	385	210§
Washington Boulevard	30	108	282	563‖
Crawford Avenue	60	60	347	¶
Crawford Avenue	60	60	357	¶
Madison Street	60	36†	593	¶
Detroit, Mich.:				
Jefferson Avenue	60	60‡	270	766‖
Clifford Street	60	32‡	400	700§
Gratiot Avenue	60	118‡	191	545§
Grand River Avenue	60	51	634	860‖

* There are no data as to whether capacity was approached in any of these instances.
† Two-car trains.
‡ Not reported in Detroit survey.[21] Data furnished by J. P. Hallihan, M. Am. Soc. C. E.
§ In one free lane.
‖ Average of two free lanes.
¶ Not recorded.

Despite the comparative meagerness of these data and the irregularity of the Chicago figures, where automobiles frequently use the car-track lane more intensively than the free lane, automobile usage seems to bear some relation to street-car frequency. The Chicago data indicate relatively free movement in the street-car lane; otherwise, more automobiles would have operated in the free lanes. The heavier flows in the car lane indicate that there may be a safety advantage in driving in the track area in comparison with the lane next to a line of parked cars.

About the best that can be deduced from these observations is the general conclusion that the operation of 120 street cars, or more, per hr, in one direction, is sufficient to require full use of the car-track area and that each reduction of 20 cars per hr below this figure will permit reasonably free movement space for 125 automobiles. Thus, a single street car, in effect, pre-empts as much street area, while in operation, as 6.25 private automobiles. Using this relation the approximate ability of a street-car lane to handle automobile traffic has been computed in Table 6 and compared with the observed data of Table 5.

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[21] "Vehicular Traffic in 1930", Detroit Rapid Transit Comm., 1930, p. 75; and "Vehicular Traffic in the Business District of Detroit", 1924, p. 18.

[22] Data furnished by E. J. McIlraith, Staff Engr., Chicago, Ill., Surface Lines.

TABLE 6.—ESTIMATED MAXIMUM PRACTICABLE, AND OBSERVED, USAGE OF CAR TRACKS BY AUTOMOBILES

Street cars per hour (one direction)	Estimated maximum practicable number of automobiles per hour in street-car lane	Observed number of automobiles per hour in street-car lane	Street cars per hour (one direction)	Estimated maximum practicable number of automobiles per hour in street-car lane	Observed number of automobiles per hour in street-car lane
0	750+	...	51	430	634
23	605	471	60	375	270, 347, 357, 568
27	580	385	90	190	...
30	560	275, 618	108	75	282
32	550	400	118	15	191
34	540	432, 506	120-150	0	...
36	525	265, 593	...	...	...

There are few data upon which to base information concerning the retarding effect of street railway cars on traffic in adjacent lanes. Traffic surveys in Detroit disclosed that the greatest flows (per lane) in the city were on streets having street cars.[20] These data are shown in Table 7.

TABLE 7.—MAXIMUM HOURLY MOTOR-VEHICLE TRAFFIC PER LANE, DETROIT, MICHIGAN

Location and nature of traffic	One-way roadway width, in feet	Lane No.	Hourly traffic
Grand River Avenue, at West Grand Boulevard (automobiles, street cars, and buses; parking prohibited)	36	1 (curb	830
		2	890
		3 (street car)	634*
Jefferson Avenue, at East Grand Boulevard (automobiles, street cars, and buses; parking prohibited)	45	1 (curb)	707
		2	895
		3	919
		4 (street car)	25†

* Also, 51 street cars; total vehicles in lane, 685.

† Private motor traffic prohibited on street-car lane, used by express street cars and local motor buses. Total vehicles in lane, 91, consisting of 41 street cars, 25 buses, and 25 private automobiles.

In view of these high records of street usage on car-line streets, which exceed those herein calculated, it seems doubtful whether any arbitrary reduction in capacity need be introduced in the present estimates because of street-car operation. If the traffic signal system is well designed, if adequate safety zones are used, if car stops are well selected, and if appropriate parking regulations are in effect, there is little reason why the lanes adjacent to the car tracks cannot carry practically the same number of vehicles per hour as similar lanes on streets without car tracks. While it is quite true that automobile speeds on street-car thoroughfares are usually lower than on automobile boulevards, because of the greater frequency of stops and the care necessarily exercised for the safety of intending passengers, the capacity of the street should not be greatly affected, as within the normal range of speeds observed on city streets (10 to 30 miles per hr) capacities are almost constant (see Fig. 1). If, however, safety zones are not used and street cars stop very frequently the loss of movement time to passenger automobiles would seriously impair the street efficiency. If this is the case, remedial measures should be applied to improve the efficiency of motor-vehicle movement. It should be noted, however, that a street with a car track has a somewhat lower estimated lane capacity than a street used only by automobiles; for example, it will be shown later that a street with an 80-ft pavement and three moving lanes in each direction (parking allowed in curb lane), and without car tracks, has an average capacity per lane of 825 vehicles in each of three lanes; that is, one "free" lane, 900; one right-turn lane, 810; and one left-turn lane, 765; total, 2 475 automobiles. (See Table 9, Items Nos. 1, 2, and 4.) If the center lane was used by street cars, the average capacity of the two remaining lanes would be 785 vehicles per hr; one right-turn lane, 810; and one left-turn lane, 765; total, 1 575. (See Table 9, Items Nos. 2 and 4.) This is an approximate reduction of 5% in the average capacity of the free lanes, due to street railway operation.

From observations in New York City, on 42d Street between Fifth and Sixth Avenues, or 14th Street between Third Avenue and Irving Place, and on Broadway between 49th and 50th Streets, the Regional Plan of New York and Its Environs[23] prepared estimates of the total reduction in the motor-vehicle capacity of these streets because of the operation of street cars. The center lane was occupied by car tracks, the next by motor vehicles, and the third by parked automobiles. The estimates are shown in Table 8 compared to the estimated reduction obtained by the application of the preceding assumptions and the use of Table 9. It will be seen that the differences are not great, and that the capacity reductions assumed in this paper are quite liberal.

TABLE 8.—ESTIMATED REDUCTION IN THE CAPACITY OF A STREET TO HANDLE MOTOR TRAFFIC DUE TO STREET RAILWAY OPERATION, NEW YORK CITY.

Location	Number of street cars per hour (one direction)	Regional Plan estimated reduction in motor-vehicle capacity, percentages [23]	Estimated reduction in motor-vehicle capacity by application of preceding assumptions, percentages
42d Street	62	26.3	33.9
14th Street	62	25.6	34.3
Broadway	33	20.9	24.8

Effect of Motor Buses on Automobile Capacity.—To determine the effect of motor buses on other vehicle traffic is a virtual impossibility with the data available. Factors affecting bus operation are more complex than those affecting rail vehicles, which are fixed in direction and location on the street surface. It is obvious, however, that just as each street car requires an area sufficient to accommodate several automobiles, so does a bus because of its greater size and frequency of stops. Surveys on Fifth Avenue in New York City in 1926 indicated that the operation of 100 buses per hr reduced the total capacity of the street by about 300 automobiles. However, the crowding on Fifth Avenue is such as to hinder bus operation greatly. For satisfactory movement, it is probable that 50% more room should be allowed, making each bus equivalent, in space occupied, to 4.5 passenger automobiles.

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[23] "Highway Traffic", by Harold M. Lewis and E. P. Goodrich, Members, Am. Soc. C. E., Regional Plan of New York and Its Environs, 1927, pp. 73–114.

Capacity of a Lane of Street Cars

Under favorable conditions a single electric railway track on an ordinary city street has a capacity in excess of 180 street cars per hr. However, to assure satisfactory speed, it will be assumed that the capacity is 150 street cars per hr, seating 60 passengers each, such as are used in city service on heavy-traffic thoroughfares.

Capacity of a Lane of Motor Buses

It is as difficult to determine the capacity of a lane of motor buses as it is to analyze their effect on other vehicle traffic. The reasons are practically the same in both cases.

Even were maximum bus capacities per lane possible of derivation, their value would be only academic because there are many considerations that would normally influence a decision to operate street cars or trolley buses in preference to motor buses when the density of passenger traffic is considerably below that which would require the maximum use of buses. Fifth Avenue, in New York City, is the notable exception. On this street more than 200 double-deck buses (seating, on the average, 56 passengers each) are operated southbound between 8:00 and 9:00 A. M. and approximately 190 per hr northbound between 5:00 and 7.00 P. M.

TABLE 9.—General Estimates of Surface Street Vehicle Capacities per Lane per Hour, for Various Conditions

Item No.	Type of lane	Conditions of use	Number of Motor Vehicles per Lane, per Hour	
			Developed by writer's assumptions	Report by Regional Plan of New York and Its Environs
1	Free lane	No turns	900	800*
2	Lane next to parked lane	Right turns	810	700–750*
3	Lane next to parked lane	Right and left turns	675	
4	Center lane	Left turns	765	
5	Curb lane	Right turns, no parking	450	
6	Curb lane	Right and left turns, no parking	360	
7	Car-track lane	30 street cars per hour	560	475–600*
8	Car-track lane	60 street cars per hour	375	
9	Car-track lane	90 street cars per hour	190	
10	Car-track lane	120 and more street cars per hour	0	
11	Motor-bus lane	30 buses per hour	675	......
12	Motor-bus lane	60 buses per hour	540	......
13	Motor-bus lane	90 buses per hour	405	......
14	Motor-bus lane	120 buses per hour	270	......
15	Motor-bus lane	150 buses per hour	135	......
16	Motor-bus lane	180 buses per hour	0	......

* General average of all lanes without reference to parking regulations.

On account of the lack of necessary data and the fact that bus operation of the character just mentioned is met with so infrequently, no bus-lane capacity will be estimated. In the later calculations of vehicle and passenger capacities of streets, however, streets with bus loadings ranging from 30 to

180 buses per hr in one direction are shown. The buses are assumed to be single-deck (suitable, for frequent stop service), of a maximum physical size considered practicable for city traffic conditions, namely, seating 40 passengers.

In Table 9, the estimated surface street capacities per lane, developed from the foregoing assumptions, are compared, as far as practicable, to those reported by Messrs. Lewis and Goodrich.[23] These values have been used in the preparation of the hourly street capacities for various widths of streets under differing conditions of use, as shown in Table 10.

TABLE 10.—AUTOMOBILE CAPACITIES OF VARIOUS STREETS UNDER DIFFERING CONDITIONS OF USE

Item No. (1)	Descriptions (2)	CAPACITY, IN MOTOR VEHICLES PER HOUR FOR ONE-WAY TRAFFIC (HALF STREET)							
		One lane		Two lanes		Three lanes		Four lanes	
		Parking prohibited (3)	Parking permitted (4)	Parking prohibited (5)	Parking permitted (6)	Parking prohibited (7)	Parking permitted (8)	Parking prohibited (9)	Parking permitted (10)
1	Express highways, no grade crossings	1 500	.....	2 670	.....	3 510	.....	3 900	......
2	Surface streets, without car tracks or buses	.....	.....	1 215	675	2 115	1 575	3 015	2 475
	Surface Streets, with Car Tracks:								
3	30 street cars per hour	.....	.....	940	420	1 775	1 235	2 670	2 135
4	60 street cars per hour	.....	.....	755	280	1 590	1 050	2 490	1 950
5	90 street cars per hour	.....	.....	570	140	1 405	865	2 305	1 765
6	120 or more street cars per hour	.....	.....	380	0	1 215	675	2 115	1 575
	Surface Streets, with Buses:								
7	30 buses per hour	.....	.....	1 025	500	1 890	1 365	2 790	2 265
8	60 buses per hour	.....	.....	910	400	1 755	1 245	2 655	2 145
9	90 buses per hour	.....	.....	795	300	1 620	1 125	2 520	2 025
10	120 buses per hour	.....	.....	680	200	1 485	1 005	2 385	1 905
11	150 buses per hour	.....	.....	565	100	1 350	885	2 250	1 785
12	180 buses per hour	.....	.....	380	0	1 215	720	2 115	1 620

The data in Table 10 are not extended beyond four lanes in each direction, nor is it believed that the assumptions herein made are applicable to streets of greater width. The table shows increases in average lane capacities as the width of the pavement increases because of the partial segregation of traffic with various objectives. On streets of greater width, it is generally believed that the effects of weaving and straddling become pronounced, with resultant decreases in the lane efficiencies. There is no authority on which to base conclusions as to the actual effect of these factors, but since urban streets with pavements accommodating more than eight lanes of traffic are seldom encountered, further consideration of this matter seems unwarranted.

STREET AREA REQUIRED PER PASSENGER IN VARIOUS VEHICLES

One of the measures of the efficiency with which a street is used by different vehicles is the street area required for each passenger transported. Usually, this is computed by dividing the static area of the vehicle by the number of passengers. A more informative method is to determine the space required for each passenger while the vehicle is in operation. This latter value may be called the operating area per passenger, and the area occupied by the moving vehicle, the operating area per vehicle.

Street Area Occupied by Various Vehicles and Their Passengers While in Motion at Approximate Capacity Conditions

The area required for the operation of an automobile may be computed from the formula:

$$A = 5\,280 \frac{VL}{N} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which, A is the operating area per vehicle, in square feet; V, the average speed, in miles per hour; L, the width of lane, in feet, assumed as 10 ft; and N, the vehicles per lane per hour.

The capacity of vehicle lanes available for movement on streets with 60 to 80-ft pavements (parking permitted) is approximately 800 vehicles per hr (see Table 10, Item No. 2). If the average speed, including stops, is 15 miles per hr, the operating area per vehicle is: $A = \frac{5\,280 \times 15 \times 10}{800} = 990$ sq ft. These computations have been based on an average over-all speed of 15 miles per hr, since it is improbable that vehicle speeds on streets with heavy traffic can be maintained at much above this rate for any sustained period of time, even if the maximum speed attained is two to three times as high. For example, surveys by the Detroit Rapid Transit Commission[24] gave average motor vehicle speeds on three heavy traffic arteries, as shown in Table 11.

TABLE 11.—Average and Maximum Motor Vehicle Speeds, Detroit, Michigan

Street	Distance, in miles	Time required, in minutes	Speed, in miles per hour, average, including effect of delays	Number of stops	Maximum speed between stops, in miles per hour
Woodward Avenue	5.0	25	12.0	27	27
Gratiot Avenue	5.0	29	10.3	34	30
Grand River Avenue	5.0	28	10.7	33	25
Woodward Avenue	12.7	46	16.5	35	43
Gratiot Avenue	12.5	47	15.9	39	35
Grand River Avenue	12.6	50	15.0	41	35

The operating area of street cars and motor buses is obtained by multiplying the area used by one passenger automobile by the number of automobiles that the street car or motor bus displaces. This is, in the case of the street car, 6 190 sq ft, and, in the case of the bus, 4 455 sq ft, since the street car and the bus have been calculated to occupy space that would accommodate 6.25 and 4.50 automobiles each, respectively.

The operating area per passenger may be obtained by dividing the vehicle area by the number of passengers in each vehicle. In these calculations use has been made, in the case of the automobile: First, of the average number of passengers observed in automobiles during the rush hour in several large cities[25]; and, second, of a full seated load in a five-passenger automobile. For the motor bus and the street car the writer has used the full seated load and also the seating capacity plus a reasonable standing load. The total passenger capacity of the street car is considered to be 90 — 60 seated and 30 standing. The motor bus is assumed to have a total capacity of 50 passengers — 40 seated and 10 standing. These standards of loading are not excessive and permit reasonably comfortable movement of passengers within the car or bus. A higher proportionate standing load is used in the street car, which because of its construction and greater width, handles standees more easily than the bus.

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[24] "Vehicular Traffic in 1930", Detroit Rapid Transit Comm., 1930, pp. 26–28.

[25] Based on surveys in Chicago, Ill., in 1929; Detroit, Mich., in 1930; Boston, Mass., in 1927; San Francisco, Calif., in 1926; St. Louis Mo., 1930; Baltimore, Md., in 1929; and Kansas City, Mo., 1930.

While it is true that the standards of loading used in the case of the individual vehicle and the mass transportation vehicle are not strictly comparable, it is a matter of common observation that the automobile is seldom loaded to its full capacity, even during the rush hour. (Only about 3 of every 100 five-passenger cars observed in traffic surveys have a capacity load.) The street car and the motor bus on the other hand, not only can be loaded to maximum capacity, but actually are so loaded ordinarily during the rush period. The street car is a service vehicle catering to large numbers of persons with many different points of origin and destination, but all along some particular route. Holding itself out for hire to all who happen to be along its route, it can serve its full seated and standing capacity just as satisfactorily as it can serve a single person.

This is not true of the automobile which is essentially a vehicle for individual service, serving most satisfactorily only the personal needs of the operator (ignoring chauffeur-driven cars). It is because of this that the average number of passengers observed in each motor vehicle is rather low. Since this figure does not vary much from city to city it seems to indicate quite definitely that, regardless of the seating capacity of the private vehicle, its practicable average rush-hour capacity does not exceed 1.75 passengers. The practicable capacity of street car and bus during rush hours, on the other hand, is approximately equal to 90 and 50 passengers, respectively. These calculations are shown in Table 12.

TABLE 12.—OPERATING AREAS PER VEHICLE AND PER PASSENGER

Vehicle	Conditions	Passengers per vehicle	OPERATING AREA, IN SQUARE FEET	
			Vehicle	Passenger
Automobile	Average rush-hour loading	1.75	990	566
Automobile	Full load	5	990	198
Motor bus	Full seated load	40	4 455	111
Motor bus	Full seated load plus 25% standing load	50	4 455	89
Street car	Full seated load	60	6 190	103
Street car	Full seated load plus 50% standing load	90	6 190	69

PASSENGER CAPACITY OF A TYPICAL STREET

A more practical and useful method of comparing the efficiency of various vehicles is to apply the previously derived capacity data to a typical street and determine its vehicle and passenger-carrying potentiality when used by individual vehicles alone and when used by individual and public vehicles.

By applying the preceding data to a street of any assumed width, its capacity—first in vehicles and then in passengers—is readily ascertainable. For the purpose of demonstration the writer has selected a street with a 60-ft pavement on which parking is prohibited and has assumed that it is used to its maximum potential ability by: (1) Automobiles only; (2) automobiles and motor buses; and (3) automobiles and street cars. The data are compared graphically in Figs. 2 and 3.

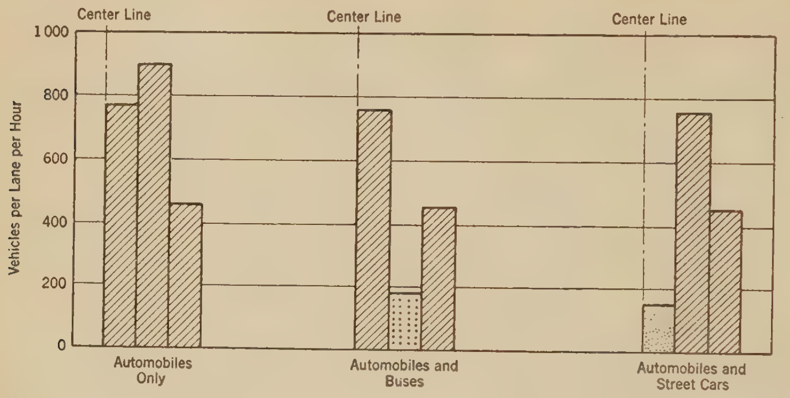

Fig. 2.—Hourly Vehicle Capacities of a Street with 60-Foot Pavement. Three Lanes on Half-Street Under Various Conditions of Use (Parking Prohibited).

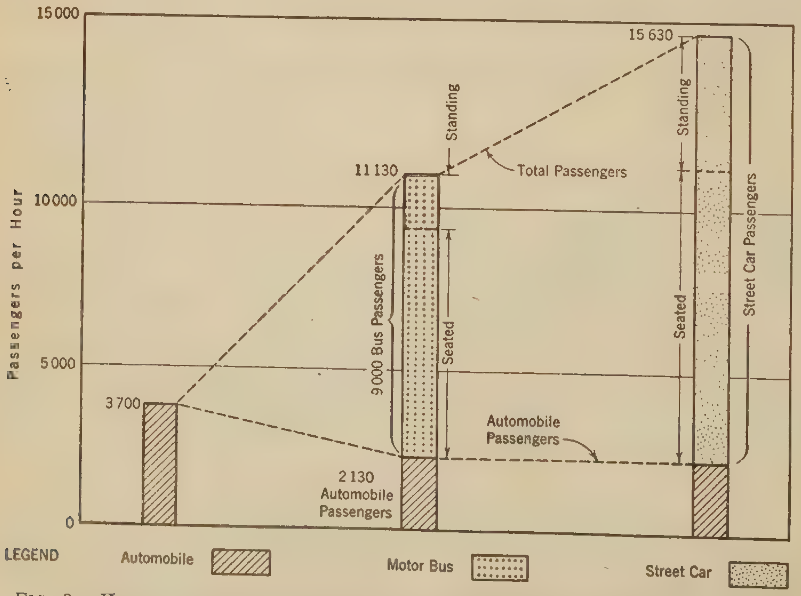

Fig. 3.—Hourly Passenger Capacities of a Street with 60-Foot Pavement Under Various Conditions of Use (Parking Prohibited).

In Fig. 3, the number of passengers is based on automobiles carrying 1.75 passengers, motor buses carrying 50 passengers, and street cars carrying 90 passengers each. These are considered to be practicable average rush-hour loads. Thus, a street of average width, used both by street cars and automobiles, has a capacity of passengers 4.25 times as great as a similar street used only by automobiles. If buses replace street cars, the capacity of the street is three times that of the automobile street.

Measures for Traffic Relief

If the results of the foregoing analysis are compared with observed traffic flows on major streets, it will be found that they are seldom being used to their maximum potential capacity. Nevertheless, congestion may exist on the same streets. Congestion is not synonymous with capacity usage, but may occur long before the street has reached its greatest usefulness. It is only an indication of inherent inefficiencies in the street system or of disorganized and inefficient usage.

When such conditions occur the application of accepted principles of traffic planning may be expected to provide substantial relief at a minimum expenditure of money. It is improbable, however, that these activities alone will always be able to provide the requisite facility of traffic flow in every part of the city. Undoubtedly, a time will come when new street facilities will be considered. Ordinarily, new and widened streets are offered as the solution. Seldom, except in the very largest cities, is much attention paid to the improvement in general traffic conditions which would follow measures devised to promote the use of public transportation systems, with their acknowledged superiority in moving masses of people into, and out of, crowded districts.

The provision of added street areas for automobile use only often fails to provide the needed traffic relief because of the stimulus to greater motor-vehicle use which frequently results from any appreciable addition to the traffic facilities. An example from Detroit well illustrates this point. Congress Street, narrow and poorly paved, leading into the central business area from the east, was widened and repaved as a relief artery for Jefferson Avenue which lies parallel to Congress Street, two blocks to the south. A cordon traffic count was made at the borders of the district in 1924 before the widening and in 1925 after the widening. Traffic volume data in 1925 compared to 1924 are given in Table 13.

TABLE 13.—Change in Traffic Volume to and from Central Business District, Detroit, Michigan

Description of streets	Percentage increase or decrease, 1924-1925
All streets into central district	10.9*
Congress Street	570.0*
Jefferson Avenue	3.0†
Congress Street and Jefferson Avenue	38.1*

* Increase. † Decrease.

The traffic decrease on Jefferson Avenue, (Table 13), is so small that it is extremely doubtful whether any material improvement in the freedom of traffic movement resulted. The high percentage increase in traffic on the combination of Jefferson Avenue and Congress Street is entirely out of line with increases in the other parts of the area, and evidences the fact that the improved street encouraged a much greater use of motor vehicles, with a continuation of about the same degree of street crowding as existed before. It does not necessarily follow that improvements of this character are wholly undesirable; in areas of limited size and excessive street crowding, reasonable increases in the street area compatible with the normal increase in the population served, are frequently quite necessary.

Conclusion

Personal transportation on city streets is furnished in private vehicles (automobiles) and public vehicles (rapid transit trains, street cars, buses, and taxicabs). All have their place in the traffic pattern. None can be excluded and no one can be sole agency for passenger movement. All must be coordinated in any comprehensive traffic plan so that each may play its appropriate and most efficient part because, despite the enormous increase in automobile ownership, collective travel continues to be a predominant factor in urban transportation.

TABLE 14.—Modes of Transportation to and from Central Business Districts

Item No. (1)	Order in size of cities in United States (1930 census) (2)	City (3)	Year of survey (4)	Percentage of Passengers Entering and Leaving by: Mass transportation vehicles (5)	Passenger automobiles (6)
(a) Population, 1 000 000, or More					
1......	2	Chicago, Ill..............	1929	78.1	21.9
2......	3	Philadelphia, Pa.........	1928	79.2	20.8
3......	4	Detroit, Mich...........	1930	66.2	33.8
4......	5	Los Angeles, Calif........	1924	65.3	34.7
(b) Population, 500 000 to 1 000 000					
5......	7	St. Louis, Mo...........	1930	65.4	34.6
6......	8	Baltimore, Md..........	1929	60.8	39.2
7......	9	Boston, Mass............	1932	65.7	34.3
8......	10	Pittsburgh, Pa...........	1927	74.6	25.4
9......	11	San Francisco, Calif......	1926	74.5	25.5
10......	12	Milwaukee, Wis..........	1926	50.4	49.6
(c) Population, 250 000 to 500 000					
11......	14	Washington, D. C.......	1930	36.7	63.3
12......	19	Kansas City, Mo........	1930	48.9	51.1
13......	24	Louisville, Ky...........	1926	53.6	46.4

The service rendered by public transportation is best measured, perhaps, by the proportion of workers and shoppers transported to and from central business districts, as summarized from available data, in Table 14.

Taxicabs are included in the values in Column (6), except in the case of Louisville (Item No. 13) where they are included among the mass transportation vehicles (Column (5)). Item No. 1, of Table 14, covers twelve hours of the business day. Of the automobiles crossing the boundary line of the central business district, more than 30% passed through without stopping for the transaction of any business. In rush hours, the percentage of through vehicles remains substantially the same, although passengers outbound in automobiles in the rush hour comprise only 11% of the total outbound passenger movement. Further analysis shows that of the total people in the central business district at any one time, never more than 7% had arrived by automobile; the others had come by means of public transportation systems.

If it is assumed that the cities in Table 14 are typical of others in their respective population classes, approximately two-thirds of all the 29 000 000 people in the cities of more than 250 000 population, are dependent on public transportation for their daily travel to and from business areas. It is noted that the proportion that uses the public carrier generally increases with the size of the city, and since the trend in National growth, at least up to 1930, has been toward the city, it is quite likely that public transportation because of its high passenger-carrying ability, may assume a greater importance than in the past. Briefly, the facts surrounding urban and rural growth are:

1.—Approximately one-fourth of all the people of the United States in 1930 lived in the 37 cities of 250 000 population and more. In 1890, less than one-eighth lived in the 11 cities in this class.

2.—One-third of the population lived in the remaining 3 128 cities of 2 500 to 250 000 population. In 1890, it was less than one-quarter.

3.—In 1930, 43.8% of the population lived in rural territory. In 1890, 64.6% of the total lived outside of urban areas.

4.—Urban population has more than tripled in the last 40 years; rural population has increased a bare one-third.

No growing city can cope successfully with the traffic problem of the present or prepare for the more complex conditions of the future unless it simultaneously carries on two different types of traffic activities: (*a*) Employing such means as are immediately available to obtain higher efficiency from existing services; and (*b*) constructing additional traffic facilities to provide for the future normal increase in traffic volume.

Neither of these types can be prosecuted to the total or even partial neglect of the other. In each instance, it is important that appropriate consideration be given to both public and private vehicles and the facilities for their operation. The amount of money expended in furnishing facilities for each type of vehicle should be in proportion to the economic importance of each in the city plan.

ACKNOWLEDGMENT

The writer is indebted to the members of the Committee on Street Traffic Economics of the American Transit Association, for valuable assistance in furnishing fundamental data bearing on street usage and for their critical comments during the preparation of this paper.

DISCUSSION

R. O. THOMAS[26], ESQ. (by letter).—A timely emphasis of the fact that city streets are not used, either economically or actually, to their greatest capacity, is afforded by this paper. The writer agrees with Mr. Simpson's data and conclusions in all respects except as to the influence of curb parking on street capacity.

Some of the results of a survey, in 1932, of the parking situation in the Central Business District of Los Angeles, Calif., may be of value in determining the extent to which street capacity is reduced by permitting parking in the lane next the curb. In this District about 37.7% of the total curb length is available for curb parking by private machines. Computed on a basis of 19.5 ft per car, there is room to park 31 000 cars at the curb during a business day. The hours during which parking is permitted for a 45-min period are from 8:00 A. M. to 4:30 P. M. This gives 31 000 ± car-periods. The writer is aware that 18 ft per car is the generally accepted curb length assigned to each car, but a series of about 1 500 measurements made directly at the curb, gives an average of 19.5 ft of curb space per car.

By actual count, the number of car-periods utilized at the curb during the day was 29 000. This discrepancy is caused by the fact that on the outskirts of the Central Business District, the curb is not utilized to its full capacity. However, by actual count, the number of cars parked, was 20 000±. These 20 000 cars, due to the fact that 5 000±, or 25%, remained overtime for one or more periods, used the space which might have been occupied by 29 000 cars parked for the legal time limit.

During the progress of the parking survey (which extended over a period of several months) attention was given to the utilization of off-street parking facilities, with results as shown in Table 15.

TABLE 15.—OFF-STREET STORAGE CAPACITY AND USE, CENTRAL BUSINESS DISTRICT, LOS ANGELES, CALIFORNIA.

Facility	Number	Capacity	Estimated turnover	Rate of turnover
Public garages	99	14 065	12 272	0.872
Private garages	71	5 461	5 181	0.948
Public automobile parks	306	32 769	40 041	1.224
Private automobile parks	117	3 998	2 531	0.632
Totals	593	56 293	60 025	1.068

The turnover in each case was found by contacting the proprietor and employees of the parking garage, station, or lot. When a car is parked in a garage, it is assumed that it is the intention of the owner to leave it there throughout the day. That leaves the public and private open-air lots to care for the transient trade. The writer will concern himself only with the public parks. It is believed and on good authority, that a daily turnover of 5.0 is not unreasonable to assume and that this figure can be easily reached by the average parking lot. Thus, with a capacity of 33 000, the parking lots could be expected to care for 165 000 vehicles per day.

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[26] Los Angeles, Calif.

The writer has gone into this subject at such length in order to provide a basis for the claim that it is not absolutely necessary, as it is frequently held, to destroy the capacity of the city streets in order to extend privilege to a favored few. It appears from the foregoing that the 20 000 cars which actually park at the curb during the average day could be adequately cared for by the parking lots. It was observed that although the total daily turnover was slightly more than the capacity of the lots, there was no time during the day when the lots were filled to anywhere near their rated capacity. The useless waste of space is tremendous.

In the Central Business District of Los Angeles, the streets are from 60 to 80 ft wide. If parking is allowed on two sides of the street, the effective width is cut to from 40 to 60 ft. Then, if a right of way is provided for two lines of street-car tracks the following situation arises: At any time a street car is traversing a block, it is impossible for automobiles to pass it because they cannot get between the row of parked automobiles on the one hand and the street car on the other. It is manifestly impossible for them to pass the street car on the left, because of the danger of collision with machines using the other side of the street. It appears, therefore, that when street cars are in a block on which parking is allowed, the private machines also using that block are necessarily limited in speed to the rate of travel of the street car. Consequently, Mr. Simpson's estimate of 5% as the reduction of capacity of the lane next the line of parked cars is too low; it should be 25, 30, or even as much as 40 per cent. Investigation along this line should prove extremely interesting.

The foregoing arguments are powerful ones for the abolition of parking in city streets. Such parking can be defended on no other basis than that of special privilege; it lessens the use of the streets to an alarming extent; it is not necessary, since parking lots are capable of caring for the relatively small percentage (about 20% of the total number of cars parked) of cars parked at the curb; and, finally, the privilege is gravely abused, since out of a possible usage of 29 000 car-periods, only 15 000 cars, or slightly more than 50%, obey the law and park only for the legal length of time.

The remainder of Mr. Simpson's conclusions are of untold value and should be consulted by every legal body prior to the drafting of a traffic ordinance.

The previously mentioned survey was performed by the writer, as Assistant, under the direction of Donald M. Baker, M. Am. Soc. C. E., who, as Consulting Engineer for the Los Angeles (Calif.) Traffic Association, conceived and carried the survey through to completion. Credit is due to Mr. Baker for permission to use the facts which were gathered during the course of the survey.

Sigvald Johannesson,[27] M. Am. Soc. C. E. (by letter).—It is evident that the author has made a careful study of available evidence in preparing this paper. An important statement in it is that "the passenger capacity of a city street is of much greater social importance than its vehicle capacity."

The indication is that the passenger capacity of a street is far greater when street cars are used for transportation, than when the passengers are transported by automobiles. The writer agrees with this in principle, although he may not agree with all the statements and assumptions made by the author.

In the matter of passenger capacity, Mr. Simpson has considered automobiles, buses, and street cars. It might be worth while in this connection to consider, also, the bicycle. The "operating area" of a bicycle, using the author's method of determining this area, may be taken to be $\frac{5\,280 \times 8 \times 3}{5\,280}$ = 24 sq. ft, as compared with 69 sq ft for the street car. On this basis the passenger capacity of a street would be nearly three times as great when bicycles are used for transportation as when street cars are used for this purpose. The question remains, however, whether the general public would favor the use of bicycles in city streets to the complete or partial exclusion of automobiles, just as the question remains whether it favors the use of street cars to the complete or partial exclusion of automobiles.

The title of the paper implies that only capacity is discussed. For a complete study of the economics involved in street transportation, it is necessary also to consider the time element.

Outside his working hours the value of a man's time is that which he himself chooses to place upon it, according to his desire and ability to pay. If he does not consider his free time to have any money value, then, within certain limits, the most economical means of transportation is walking. This means of transportation was once quite common, and is still largely used in places where ready cash is scarce. The reason why walking as a means of transportation, say, within a radius of 3 or 4 miles, is not generally accepted as formerly is principally, that people prefer to save transportation time and are able and willing to pay for this saving. For example, a man living within 2 miles of his place of business could walk this distance in 30 min, but prefers to take a street car, in which he can cover the distance in 10 min, saving 20 min for an outlay of perhaps 7 cents for carfare. Under these circumstances he considers, apparently, that the value of his own time is at least $\frac{1}{3}$ cent per min.

As a rule, the general public places a money value on time and for that reason desires reasonably fast transportation and is willing to pay to obtain it. Quite properly the author states that the presence of street cars reduces the rate of speed at which automobiles can be operated in city streets, and an estimate may readily be made of the money value of the time lost due to this reduction of speed. The important question, however, is not whether street cars, buses, or automobiles afford the greatest passenger capacity, but whether the general public prefers a large "passenger capacity" rather than speed of travel. The former can be obtained most readily by walking or bicycling, and the latter by the use of automobiles unimpeded by the presence of street cars.

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[27] Designing Engr., State Highway Comm., Trenton, N. J.

W. W. CROSBY,[28] M. AM. SOC. C. E. (by letter).—The writer has found this paper extremely interesting, and he considers it valuable for careful study and reference. One of its great merits lies in its suggestions of ideas for further discussion and development.

In the "Synopsis," Mr. Simpson points out that street improvement projects as well as traffic regulation plans should be developed so as to facilitate the movement of private as well as public vehicles to the greatest possible extent. Later, under the heading, "Introduction," he defends his view that the passenger capacity of a city street is of much greater social importance than the vehicle capacity, defining precisely his concept of the meaning of "capacity" under the heading, "The Meaning of Street 'Capacity'."

American views have always been toward freedom of action of the individual, including probably the individual in his motor car. Whether the National Industrial Recovery Act (NRA) will bring about a permanent change of this ideal remains to be seen. For the present, however, ideas concerning passenger and vehicle rights, such as the preceding, invite careful consideration as to their general acceptance, with or without modifications.

Among the "Factors Affecting Capacity of Multiple-Lane Surface Streets," Mr. Simpson mentions bus traffic as a major one. This brings to mind that one can walk up Fifth Avenue, in New York City, from 30th to 55th Streets, or beyond, more quickly than one can travel by bus. In this case, the effect of the buses on the other traffic seems to be greater than that estimated by the author. The allowances suggested for the reductions in capacities by turns and by parking are at least conservative and perhaps should be doubled.

Mr. Simpson under-estimates the reduction of street capacities for motors by the presence of street cars under ordinary conditions. A statement that deserves emphasis is (see "Measures for Traffic Relief") that when the time comes for considering new street facilities, ordinarily, new and widened streets are offered as the solution. It is true as the author points out that the provision of added street areas for automobile use only often fails to provide the needed traffic relief; but in that case the writer does not believe that provisions for "mass transportation" are always indicated.

The writer agrees, generally, with the advanced conclusions, but would suggest, with respect to the last sentence under "Conclusion," that the money expended should be in proportion to the social and recreational importance, as well as the economic importance, of each type of vehicle in the city plan. Even when considering large cities, it must be admitted that people live in them, who do not merely move from homes to work and back.

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[28] Cons. Engr., Coronado, Calif.

HAROLD M. LEWIS,[29] M. AM. SOC. C. E. (by letter).—This paper should be of great assistance to city engineers and city planners in arriving at a logical design of the roadways in city streets. There has been considerable confusion as to the capacity of streets and highways in terms of the number of vehicles they can carry. This has resulted largely from the use of the term, "vehicles per hour," as the designation of traffic capacity whether the figures used actually represented those passing within an hour or the average rate of vehicles within either shorter or longer periods. Much of this confusion would be eliminated if this term were not used when it was desired to express the intensity of traffic for periods of less than one hour.

Some of the lane capacity curves in Fig. 1 are based on the assumption that it would be possible to have traffic throughout a full hour of the intensity that has been found for periods of only a few minutes. It is just as illogical to consider such short period intensities as representing a possible hourly street capacity as it is to consider the tops of the waves the theoretical level of the ocean. Traffic also moves in waves, and he who is trying to determine the necessary width of a roadway should design it for something less than the intensity of a maximum wave of traffic. It is no more possible to fill up the space between these waves of traffic and still maintain rapid rates of speed, than it is to raise the ocean to the level of the tops of the waves.

The maximum number of vehicles that may reasonably be expected to pass a given point within one full hour is a reasonable basis for determining the width of a street or thoroughfare. At special locations, such as busy intersections and at the approaches or exits to bridges or tunnels, those intensities which may be found over shorter periods should be made a basis of local design. The maximum number of vehicles that might be expected within a 15-min period should furnish a basis for such designs.

Curve 9 in Fig. 1 was developed by the Regional Plan of New York and Its Environs, and is lower than the others because it was based on traffic that may be expected within a full hour and because of the more general use of four-wheel brakes since it was developed. It was presented in the Regional Plan report on "Highway Traffic"[9] as "a basis for designing future streets and checking existing capacities" in a single lane of traffic, and it was pointed out that a roadway with two or more lanes in each direction would have lower average lane capacities. The curve was based on observations made in 1924 of spacing between groups of vehicles. The universal use of four-wheel brakes since that time has made it safer for vehicles to drive closer together at the higher speeds, which would tend to decrease the drop in this curve after it reached its peak. Such a revision would make it somewhat similar in general shape to Curves 7 and 8, in Fig. 1.

Mr. Simpson has given a value of 900 vehicles per lane per hour as the capacity of a traffic lane devoted to straight-through movement on a surface street. While it is undoubtedly possible for traffic to reach such values in isolated cases, a better figure to use for the purpose of street design is 750 vehicles per lane per hour on streets with two or three moving lanes in each direction. This is 50% of the maximum capacity of a single lane on an express highway, which the writer agrees with Mr. Simpson is about 1 500 vehicles per hr.

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[29] Cons. Engr.; Engr., Regional Plan Assoc., Inc., New York, N. Y.

During October and November, 1924, the Regional Plan Staff made a total of 983 observations of the volume and speed of vehicles on typical streets in the Borough of Manhattan. The velocity, including necessary stops for traffic, was found to vary from 2 to 25 miles per hr. The weighted average of all the observations was 11.3 miles per hr, and it was found that with this velocity many of the streets were carrying only 150 to 500 vehicles per moving lane per hr.

TABLE 16.—NUMBER OF PERSONS PER CAR IN VEHICLES, EXCLUDING BUSES, ENTERING MANHATTAN SOUTH OF 59TH STREET ON A TYPICAL BUSINESS DAY IN JUNE, 1932.

Location	In passenger cars	In trucks
On Avenues between 58th and 59th Streets:		
First Avenue	1.62	1.42
Park Avenue	1.64	1.35
Fifth Avenue	1.61	....
Seventh Avenue	1.94	....
Ninth Avenue	1.29	....
Tenth Avenue	1.41	1.43
Columbus Circle	1.53	1.34
On upper roadway of Queensboro Bridge	1.69	....
Average observations crossing 59th Street	1.59	1.39
Holland Tunnel	1.88*	1.43*
From 23d Street Electric Ferry, across Hudson River	1.62*	1.52*
From 42d Street Ferry, across Hudson River	1.81*	1.58*
Adopted as a general average	1.7	1.4

* An average of several half-hour observations.

In 1932, the Regional Plan Association made some traffic counts to determine the number of persons and vehicles entering Manhattan south of 59th Street on a typical business day. In this connection a series of counts of persons per vehicle were made. They may be of interest as supplementing Mr. Simpson's figures. Such counts were made over half-hour periods and included the drivers of the cars in each case. The results are given in Table 16.

TABLE 17.—PERSONS ENTERING MANHATTAN SOUTH OF 59TH STREET, NEW YORK CITY, ON A TYPICAL BUSINESS DAY, IN JUNE, 1932

Means of transportation	Number	Percentage
By mass transportation vehicles (buses, trolleys, or rapid transit)	1 976 419	75.1
In other vehicles (private cars, taxicabs, and trucks)	494 408	18.8
Pedestrians on bridges and ferries	159 805	6.1
Total on city streets	2 630 632	100.0
By railroad	161 180	....
Total entering Southern Manhattan	2 791 812	....

This same traffic count, together with statistics regularly kept for traffic on the ferries and bridges to Manhattan, has made it possible to compile figures for the City of New York comparable with those quoted in Mr. Simpson's Table 14, showing modes of transportation on city streets to and from central business districts. It was found that of all persons entering Southern Manhattan across 59th Street, excluding railroad passengers to the Grand Central Terminal, 74.0% used mass transportation vehicles (buses, trolleys, or rapid transit) and 26.0% used other motor vehicles (private cars, taxicabs, or trucks).

The modes of transportation used by all persons entering that part of Manhattan south of 59th Street, from all directions, on a typical business day are as shown in Table 17.

HAWLEY S. SIMPSON,[30] ASSOC. M. AM. SOC. C. E. (by letter).—The writer has been especially interested in the discussion of this paper for the reason that the comments have concerned the social and economic phases of street use more than the purely technical problems involved. A clearer understanding, by public officials, of the appropriate distribution of street space among the several classes of vehicles, on a sound economic basis, should do much to expedite measures for improving street efficiency. Such an allocation presupposes an evaluation of the worth, to the public generally, of the service rendered by each transportation medium. Too often, the streets of the city are looked upon merely as common property to be used in whatever way desired by the individual who arrives first on the scene. Too little consideration is given to the important public service which the street performs and the detrimental effect of permitting the streets to be used too freely for purposes only incidental to the act of transportation.

Mr. Thomas emphasizes his point clearly in commenting upon the gross inequity of permitting an almost negligible proportion of street users to preempt so great a proportion of curb space for long-time parking. The long lines of parked and stored automobiles seen on every important business street clearly indicate a lack of understanding of the primary use to which a street system should be put. Gradually this condition is being remedied and it is not too far-fetched to anticipate that the time will soon come when all parking will be eliminated on important thoroughfares, and in central business districts. It may be that the privilege of stopping even momentarily for receiving and discharging passengers will be prohibited during periods of congestion and this function of the act of transportation relegated to near-by side streets. Economics demand the adoption of such regulations in many places already, and eventually the public will insist upon the use of the full width of streets for movement purposes.

Mr. Crosby properly suggests that the money expended in furnishing facilities for public and private vehicles should be in proportion to their "social and recreational" importance as well as their "economic" importance. With this the writer agrees, except for one reservation, namely, that the economic needs of a city should take precedence over social and recreational demands. As Mr. Crosby states, cities are made up of more than people who only travel to and from their places of business. Nevertheless, cities are primarily commercial and industrial machines and if those thoroughfares which provide the arteries of communication for purposes of commerce and industry are subordinated, the city becomes an inefficient economic entity. No sum of money spent on measures for social and recreational transportation, regardless of how desirable they may be, will compensate for the economic losses occasioned by an improperly organized network of utilitarian thoroughfares.

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[30] Research Engr., Am. Transit Assoc., New York, N. Y.

It is probably quite true, as Mr. Crosby goes on to say, that the writer under-estimated the reduction in street capacity because of the presence of street cars "under ordinary conditions." The writer was concerned mainly, however, with the most efficient performance of a street under the kind of regulations that should be in effect if a street were to be used to its greatest potential capacity. One of the most important reasons why street cars unnecessarily congest traffic is that traffic regulations, particularly parking rules and signal systems, have often been designed without full consideration of the use to which a street is put. Adequate parking regulations, appropriately designed, progressive traffic signal systems, permitting approximately non-stop movement of both street cars and automobiles at their different normal rates of speed, together with safety zones at heavy loading points, will do much to improve the fluidity of traffic movement, and to eliminate delays chargeable to street-car operation. Without question, however, the conventional street car falls far short of doing what it could to minimize the delays inherently caused by a public service vehicle making frequent stops. This condition may soon be remedied, because the transit industry, through the Electric Railway Presidents' Conference Committee, has been engaged since 1930 in the design of a new street car involving an expenditure of more than $500 000. The new car will have higher rates of acceleration and deceleration than the best passenger automobile, an improved free-running speed, will be practically noiseless, and will be designed so as to speed up passenger loading and unloading materially.

The writer finds himself unable to agree with Mr. Johannesson's statement that the important question is, not whether street cars, buses, or automobiles afford the greatest passenger capacity, but whether the general public prefers a large "passenger capacity" rather than speed of travel. Admittedly, the individual (which, undoubtedly, Mr. Johannesson means when he uses the word "public") is mainly interested in speed. Mr. Johannesson suggests that, for a complete study of the economics involved in street transportation, it is necessary to consider the time element which was, for the most part, ignored in the paper. Speed and capacity do bear a close relation to each other, but Mr. Johannesson regards the question more from a point of view of the State highway engineer, than from that of the urban traffic official, whereas the writer's analysis dealt solely with city traffic conditions. It is quite true that the main concern of the State highway engineer is speed, or more exactly, the movement of comparatively small numbers of people over long distances at high rates of speed. The city traffic official's problem is to move great masses of people over comparatively short distances at reasonable speeds—speeds which, however, when measured in terms of those achieved on the open highway, seem snail-like. This essential difference between the rural and urban traffic problem is too often ignored in discussions of highway economics.

Mr. Johannesson's unqualified statement that speed can best be obtained "by the use of automobiles unimpeded by the presence of street cars," is most incorrect under the conditions considered in the paper. To the contrary, it is only because of the existence of the street car that automobile speeds in cities are as high as they are. This is clearly evident if one were to select, for analysis, a typical 60-ft roadway on which parking is prohibited, and which is carrying its maximum load of street cars and passenger automobiles during the rush hour. Assuming that the passenger demand is evenly distributed throughout this hour and that the average passenger trip is 5 miles, one finds that the average time required for each passenger to reach his destination is approximately 29 min. If it were not for the street car, and if all passengers had to be carried in passenger automobiles, despite the higher inherent speed of the automobile, the average travel time per passenger would increase to 75 min, and it would take exactly 3 hr, rather than 1 hr, to clear the district. Thus, the argument returns to the original thesis of the paper that, on city streets, the capacity of a thoroughfore to handle traffic in large quantities is of greater importance than the speed of the individual unit.

Mr. Johannesson's suggestion that the greatest passenger capacity may be obtained by walking, while not exactly correct, is a fact of which city traffic officials are well aware. It has been found that, when exceptional concentrations of persons must be dealt with, complete prohibition of all except pedestrians and mass transportation vehicles in the area concerned is the only method by which the masses of the people may be handled. Traffic control within the World's Fair Grounds, at Chicago, Ill., during 1933 is an appropriate illustration.

Mr. Lewis introduces some very useful data and calls particular attention to the need for designing streets on the basis of short-period, maximum traffic flows at special locations where congestion is likely to develop. The use of general hourly capacities at such places is quite improper, of course, and the greatest caution should be exercised in using general information of the kind developed in this paper at such critical locations. The expenditure of a few dollars in engineering studies at the places mentioned by Mr. Lewis will often prevent wasteful extravagance in design, on the one hand, or the construction of inadequate facilities on the other, either of which results in serious economic losses.

All those who discussed this paper call attention to certain points wherein they believe the writer's assumptions or conclusions are subject to modification. This fact suggests clearly the importance of further, and more detailed, studies along these lines, in order that both engineers and public officials may know more accurately the ability of a street system to perform its functions efficiently under varying conditions of use. Such information will make possible much more intelligent planning of traffic regulatory programs than is possible at present.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1881

STABILITY OF STRAIGHT CONCRETE GRAVITY DAMS

By D. C. Henny,[1] M. Am. Soc. C. E.

With Discussion by Messrs. H. deB. Parsons, A. A. Eremin, Calvin V. Davis, William P. Creager, F. W. Hanna, Lars R. Jorgensen, I. M. Nelidov, Paul Baumann, Thaddeus Merriman, Ivan E. Houk, A. V. Karpov, L. F. Harza, Edward Godfrey, F. Knapp, S. H. Woodard, A. Floris, Joseph Jacobs, Robert E. Glover, Charles Terzaghi, and D. C. Henny.

Synopsis

The necessity for abandoning the middle-third theory and the sliding factor as useful elements in masonry dam design is demonstrated in this paper. The writer develops the fact that the safety of straight masonry dams is principally due to shear, and he investigates the effect of loading on shearing strength. A résumé is given of available data on uplift pressure and estimates of effective uplift area and of uplift force are presented. Stability of dams of ideal triangular cross-section is studied and a shear safety factor is proposed for use in dam design.

The Levy requirement is found to be excessive for dams less than 500 ft in height under most adverse conditions of uplift, and more logical requirements fitting these conditions are suggested. Numerical examples are given of shear safety factors on the basis of cross-sections of existing dams. It is shown that the principles developed for straight concrete dams can be made equally applicable to other types of masonry dams.

Historical Review

The middle-third method of finding safe dimensions of a gravity dam was commonly used until the beginning of the Twentieth Century. It merely

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Note.—Published in September, 1933, *Proceedings*.

[1] Cons. Engr., Portland, Ore.

contemplated absence of tension in the body of the masonry near the down-stream face with reservoir empty and near the up-stream face with reservoir full.

Compliance with this requirement was deemed to afford a factor of safety of two against overturning, assuming loading stresses to be moderate. When it became evident that failure of solid gravity dams was generally accompanied by down-stream mass movement and that this was facilitated if not induced principally by penetration of pressure water at the base of the dam, or in the foundation, two new factors were taken into account, namely, uplift and sliding.

As early as 1882, danger from uplift was recognized and was considered in the design of the Vyrnwy Dam[2] of the Liverpool (England) Water System built in the period from 1882 to 1890. This dam was given a complete system of rock drains for the special purpose of reducing uplift.

The failure of the Austin (Tex.) Dam, in 1900, called forcible attention to the danger of sliding on foundation strata. It was realized that percolating water reduces friction, particularly when it is under pressure.

The first American dam in the design of which allowance was made for uplift is the Wachusetts Dam, in Massachusetts (1900–06). However, no drainage system was installed and the location of the up-stream cut-off in this as well as in the New Croton Dam, in New York State, is far down stream from the heel.

The Olive Bridge Dam, in New York State (1908–14), like the earlier Tytam and Cataract Dams, has drains in the masonry, but lacks foundation drainage.

Large dams in the United States, first provided with foundation and masonry drains to reduce uplift, are believed to be the Medina Dam, in Texas[3] (1911–12), and the Arrowrock and Elephant Butte Dams of the United States Bureau of Reclamation, in Idaho and New Mexico, respectively (1914–15). Foundations were drained to a considerable depth by drilled holes. Nearly all the high masonry gravity dams built in the United States since 1915 are provided with foundation and masonry drains.

The next step in dam design was due to experience which showed that drainage could not be depended upon to relieve, completely, the uplift under and in the body of a masonry dam. Allowance was made, therefore, in the design for both drainage and uplift and the following equation was developed:

$$F_s = \frac{P}{W - U} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which, for a unit length of dam, F_s is the sliding factor; P, the horizontal water pressure; W, the weight of masonry above an assumed sliding plane; and U, the uplift force under the sliding base. It was generally considered satisfactory if F_s was well below the coefficient of friction of masonry on itself, or on rock, the value of which was taken at a figure close to 0.75 in the case of hard rock foundation.

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[2] "Design and Construction of Masonry Dams," by Edward Wegmann, M. Am. Soc. C. E., Seventh Edition, p. 82.

[3] *Engineering Record*, December, 1912, p. 632.

A further step is proposed in this paper, namely, the consideration of shear resistance rather than sliding resistance in ordinary cases. The basic formula proposed is:

$$Q = \frac{S}{P} \qquad (2)$$

in which, Q is the factor of safety in shear; S, the total shearing strength that must be overcome before down-stream movement of any part or the whole of a dam can take place; and P, the water pressure on the projected area of the part assumed to move, on a vertical plane normal to the direction of motion.

SHEARING STRENGTH

Shearing strength is difficult to determine because in most shear tests there is interference from bending and resultant tension. There is likewise uncertainty regarding the distribution of shear. In addition, the important question arises as to whether shearing strength along any plane of movement is affected by the loading normal to that plane. The internal friction theory, originated by Coulomb, assumes that resistance to movement consists of two parts: (*a*) The pure shear strength encountered when there is no load on the plane of shear; and (*b*) an addition in the nature of friction, as expressed by:

$$s = s_1 + kq \qquad (3)$$

in which, s is the total unit shearing strength; s_1, the strength under conditions of no load; q, the unit load on the plane of shear; and k, the factor of shearing strength increase (assumed by Coulomb to be constant).

This theory has not been proved. The conception, however, of an increase in shearing strength due to compression has been accepted by many engineers, partly on the basis of experiments and partly on theoretical reasoning. It is argued that compression lessens the distance between molecules, thereby increasing their mutual attraction and making relative movement more difficult.

It has also been reasoned in connection with the classical cubical unit element that shear induces diagonal tension which, in brittle material, may be the main cause of shear failure, and that this tension is diminished by mass compression.

Punching shear tests made by A. N. Talbot, Past-President and Hon. M. Am. Soc. C. E., in 1906,[4] gave results with concrete of 1:3:6 proportions, about 60 days old, as follows:

Slabs	Shearing strength, in pounds per square inch
A	936
B	1 025
C	1 688

The specimens were 13 by 13-in. concrete slabs, 3 in., 5 in., and 5 in. thick, respectively. Slab *B* was a recessed slab and thirty-two of these were punched. Slab *C* was of reinforced concrete.

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[4] "Tests of Concrete," *Bulletin No. 1*, Eng. Exp. Station, Univ. of Illinois, Urbana, Ill.

The tests with Slabs *A* and *B* caused the formation of four radial cracks in the concrete surrounding the punch hole, and fine cracks apeared in the reinforced slabs, *C*. The formation of these cracks proves that tension was set up in the outer portion of the slabs and that inward pressure was exerted radially against the outer cylindrical face of the punched-out disk. With the 5-in. recessed slabs (*B*), this pressure must have been greater than with those 3 in. thick (*A*), and it must have been still greater with the reinforced slabs (*C*). Correspondingly, with progressively greater pressure on the disk, higher values of shearing strength were found.

Great differences were found between the compressive strength observed on the tests of 6-in. cubes (2 075 lb per sq in.) and that apparently resulting from the tests of 8 by 16-in. cylinders (1 241 lb per sq in.). This is due, as is well known, to the fact that the breaking of a cylinder of the usual proportions is not basically a compression or crushing test; in reality, it is a structural test, measuring the strength of a concrete column, failure being due usually to diagonal shear. For this reason that class of test should yield information regarding shearing strength.

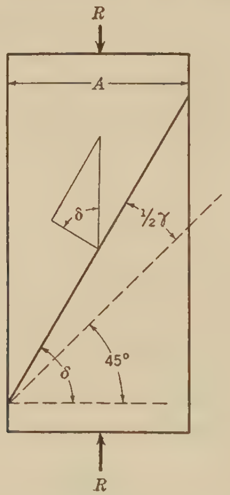

FIG. 1

Fig. 1 illustrates a plane of breakage in a cylinder test. Assuming the correctness of the Coulomb theory, it can be shown[5] that, in Equation (3), with $\frac{R}{A} = r$:

$$s_1 = \frac{r}{2 \tan \delta} \qquad (4)$$

and

$$k = \tan \gamma \qquad (5)$$

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[5] "Strength of Materials," by the late George F. Swain, Past-President and Hon. M. Am. Soc. C. E., First Edition, pp. 118–119.

Since $\gamma = 2\ (\delta - 45°)$, the measurement of r and δ permit the determination of s_1 from Equation (3), provided the break occurred along a reasonably even shearing plane as in Fig. 1, or the more common cone break.

The angle of break in concrete is usually between the limits of 60 and 70 degrees. Values of k and s_1 are shown in Table 1.

TABLE 1.— VALUES OF k AND s_1

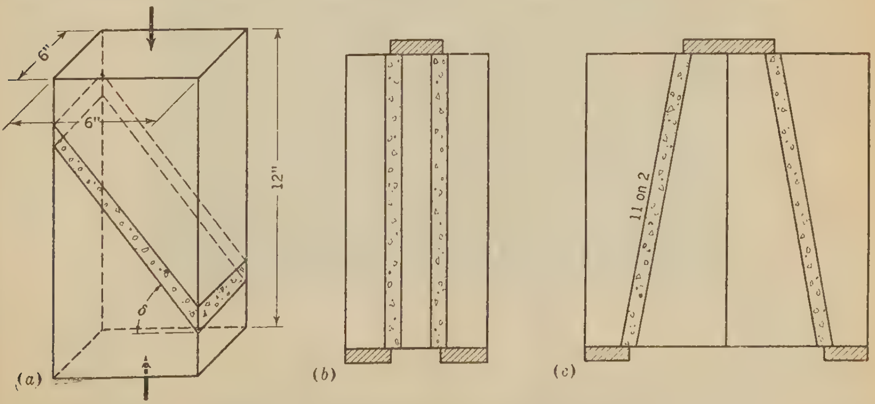
δ	γ	k	$\frac{s_1}{r}$	s_1 for $r = 3\ 000$ lb per sq in.
60°	30°	0.58	0.289	867
62° 30′	35°	0.70	0.261	783
65°	40°	0.84	0.234	702
67° 30′	45°	1.00	0.207	621
70°	50°	1.19	0.182	546

Thus, if the angle, δ, of a 1 : 3 : 6 cylinder, 60 days old, measures 62° 30′ with $r = 3\ 000$ lb per sq in., s_1 will be 783 lb per sq in., and $k = 0.70$; from which $s = 783 + 0.70q$.

TABLE 2.—TESTS OF CONCRETE CYLINDERS, UNITED STATES BUREAU OF RECLAMATION

Number of specimens	Size of cylinders, in inches	Concrete mixture	Maximum size of aggregate, in inches	Wet screened to (inches):	Age, in days	BREAK		$k = \tan\gamma$	r, in pounds per square inch	s_1, in pounds per square inch
						Kind	δ			
3	6 by 12	1:9.5	..	1½	28	Cone	73° 42′	1.56	4 300	670
3	6 by 12	1:9.5	..	1½	28	Diagonal	60° 30′	0.60	4 260	1 200
3	8 by 16	1:9.5	..	3	28	Cone	68° 18′	1.06	3 540	700
6	8 by 16	1:9.5	..	3	28	Diagonal	63° 42′	0.76	3 540	870
2	8 by 16	1:9.5	..	3	90	Cone	66° 30′	0.93	4 470	970
1	8 by 16	1:9.5	..	3	90	Diagonal	67°	0.97	3 880	820
2	12 by 24	1:7.63	3	..	28	Cone	67°	0.97	3 490	740
1	24 by 36	1:8.92	6	..	28	Cone	68°	1.03	3 020	610
3	36 by 72	1:9.5	9	..	28	Cone	63° 18′	0.74	3 050	770
1	36 by 72	1:9.5	9	..	90	Diagonal	67°	0.97	3 810	810

In 1932 some breakage tests were made in the laboratory of the U. S. Reclamation Bureau, at Denver, Colo., with cylinders of various sizes, ages, proportions, and maximum aggregates. The results are shown in Table 2.



(a) (b) (c)

FIG. 2.— SPECIMENS TESTED BY S. H. WOODARD, M. AM. SOC. C. E.

It will be noted that there is considerable variation in the values found, as might be expected from the relatively small number of specimens tested and especially from the uncertainty of the effect of restraint at the ends.

Interesting experiments were made by Silas H. Woodard, M. Am. Soc. C. E., in 1925, with mortar specimens (see Fig. 2(*a*)), which were broken at 28 days under vertical loading. They were made of 1:1 cement mortar, except that a $\frac{3}{4}$-in. zone of 1:3 mortar passed through each specimen at different angles with the horizontal. The object was to insure failure under load by shear in the weak zone and thus to ascertain the relation between shearing strength and loading on the plane of shear. In all cases breaks occurred in the weak zone. The component, $R \sin \delta$, of the breaking load, R (δ being the angle of the zone with the horizontal), was taken as the shear load, and $R \cos \delta$ as the loading normal to the plane of shear, in studying the results of Mr. Woodard's findings. The breaking loads, R, for the various values of $\tan \delta$, are given in Columns (2), (5), and (8) of Table 3, which, in Columns (3), (6), and (9), gives the shearing strength, s, for normal loading, q, both in pounds per square inch, in the manner explained previously.

TABLE 3.—SHEARING STRENGTH, s, UNDER DIFFERENT NORMAL LOADS, q.

Tan δ	FIRST SERIES			SECOND SERIES			THIRD SERIES		
(1)	R (2)	s (3)	q (4)	R (5)	s (6)	q (7)	R (8)	s (9)	q (10)
0.40	154 760	1 480†	3 710†	145 650	1 390	3 480	156 270	1 500†	3 740†
0.80	141 170	1 920	2 400	138 850	1 880	2 340	141 070	1 920	2 300
1.20	98 320	1 345	1 102	115 170	1 613	1 301	95 210	1 330	1 076
1.75	90 300	1 070	615	86 340	1 030	590	99 000	1 180	680
2.33	81 880	954	414	99 700	1 180	502	91 350	1 180	460
∞	98 190*	682	0	100 640*	706	0	101 200*	690	0
—5.50	78 480	527	—96	57 650	384	—70	66 670	444	—81

* Double actual load, specimen breaking in one zone only.
† Specimens did not break.

The values of s in Table 3 were plotted against values of q, for each of the series, and a smooth curve was drawn through these points. Then, points were read from the curve and recorded in Table 4. It appears from these experiments that the value of k is not constant, and in that respect these test results do not agree with the theory of Coulomb. The important fact is evident however, that for 1:3 mortar there is a persistent increase of shearing

TABLE 4.—VALUES OF FACTOR, k, SHEARING STRENGTH INCREASE DUE TO LOAD, q.

Load, q, in pounds per square inch	Shearing strength, s, in pounds per square inch	Shearing strength increase, kq, in pounds per square inch	k
—100	400	—290	0.35
0	690	0	0.74
500	1 060	370	0.74
1 000	1 350	660	0.66
1 500	1 580	890	0.60
2 000	1 770	1 080	0.54
2 500	1 940	1 250	0.50

strength with increasing load; that within limits of loading from 0 to 1 000 lb per sq in., k is fairly constant and may be taken at 0.70; and that the shearing strength of the mortar tested may be computed as $690 + 0.70q$.

The strength of 28-day concrete test cylinders (4 by 8 in.) has been measured at the University of Illinois,[6] under conditions of axial and tri-axial loading. The mix was 1:2.1:2.5. The breaking load was applied axially and the circumferential load was produced by oil pressures ranging from 180 to 4 090 lb per sq in. This pressure appeared generally to prevent the occurrence of a shear break, but a clean break resulted in one case (Specimen 114). The breaking load was 2 840 lb per sq in., the oil pressure, 180 lb per sq in., and the shear angle, 65° 17′. From this one experiment, a value of $s = 649 + 0.664q$, would result. Single experiments of this type can have but little weight; yet the one quoted herein conforms satisfactorily to the theorem that shearing strength increases with load.

Near the base and along the abutments of a dam the shearing strength of the foundation rock is involved, in regard to which little is usually known. Tests with granite, limestone, and sandstone in building blocks have shown strengths ranging from 1 000 to 2 800 lb per sq in.

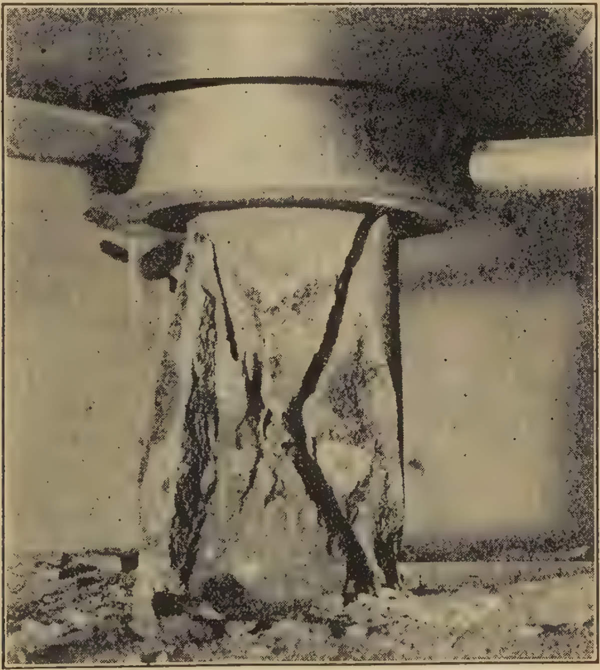


FIG. 3.—COMPRESSION TEST OF DRILL CORE; SPECIMEN OF ANDESITE TUFF BRECCIA FROM ABUTMENT OF BOULDER DAM.

Compression tests with cylinder specimens of rock may give some information. Breaking tests of drill cores taken from the rock at the abutments of Boulder Dam may be cited. The cylinders were 4.75 in. in diameter and were cut 9.5 in. long. Fig. 3 shows a broken sample in the testing machine and Fig. 4 shows parts of eight other samples. The angle of the break in Fig. 3 is 66° and, in Fig. 4, it varies from 65 to 70 degrees. The minimum load at failure for a thoroughly soaked specimen was 11 700 lb per sq in., and the average was 17 000 lb per sq in. Using the lower value (11 700 lb per sq in.), Table 5 was prepared on the basis of Equations (4) and (5).

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[6] *Bulletin No. 185*, Eng. Exp. Station, Univ. of Illinois, pp. 56, 57, and 59.

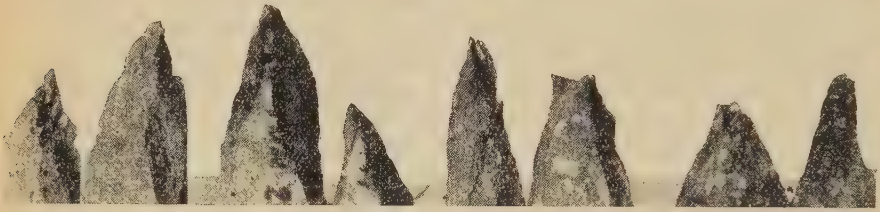

Fig. 4.—Tests of Rock Samples (Andesite Tuff Breccia), Bored from Abutment Foundation, Boulder Dam.

The experiments herein quoted are by no means conclusive as to safe values of shear. It is remarkable (and must be seriously regretted, since shear is a common cause of failure in masonry), that there is such a great paucity of published data on this subject. Additional laboratory tests have been suggested, such as torsion tests under variable axial loading. For the purpose of this paper, the broad truth of the Coulomb theory will be assumed under conditions of compression loading not exceeding 1 000 lb per sq in.

TABLE 5.—Values of s_1 in Andesite Tuff Breccia Rock Cores, Boulder Dam

Angle of shear break with horizontal in degrees, δ	Factor of shear increase, k	Cylinder test; compressive strength, in pounds per square inch, r	Shearing strength, in pounds per square inch, under no load, s_1
65	0.84	11 700	2 740
66	0.90	11 700	2 610
70	1 20	11 700	2 140

Thus, taking the correctness of Equation (3) for granted and combining it with Equation (2):

$$Q = \frac{s_1 + kq}{P} \text{ (base area)} = \frac{S_1}{P} + k\,\frac{W - U}{P} \quad\ldots\ldots\ldots\ldots(6)$$

in which, S_1 is the full base shearing strength under no load condition. Equation (6) is predicated on unimpaired intimate contact and adherence over the entire base.

Uplift Pressure

In ordinary cases of good rock foundations the pressures at the base of the dam decrease in the down-stream direction so that the pressure gradient falls well below a straight line that extends from full reservoir head at the up-stream face to zero at the toe. This has been shown[7] convincingly by

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[7] *Civil Engineering*, Vol. 2, No. 9, September, 1932, p. 578.

Ivan E. Houk, M. Am. Soc. C. E. It was also found that the pressure at concrete construction joints was quite small, and, in some cases, could not be measured.

The average uplift pressure under existing dams has been reported[8] as follows:

Dams.	Average pressure, in feet, in terms of reservoir head, H Drained	Undrained
Neye, Germany	.....	0.349 H
Brule River, Wisconsin	0.336 H	.....
Medina, Texas	0.290 H	.....
Gibson, Montana	0.248 H	.....
Willwood, Wyoming	.....	0.246 H
Pit River, California	0.190 H	.....
American Falls, Idaho	0.153 H	.....
Ariel, Washington	0.121 H	.....
Bull Run, Oregon	0.097 H	.....

The direct effect of drainage on uplift pressure is difficult to determine. At times, it is very material as was shown by the test closing of a rock drain at Ariel Dam, which caused the pressure to rise at a point 16 ft from the water face from 0 to 80% of the water height.

Drains are likely to close and become clogged. They should be straight and accessible for periodical cleaning, but there is no certainty that this will not be neglected. Conservatism in design dictates that no dependence should be placed on the perpetual reduction of uplift pressure by drains. It is proposed, therefore, in estimating uplift pressure to use an even gradient from H at the water face to 0 at the down-stream face. This is equivalent to an average pressure along any horizontal plane of $0.50H$, which is seen to be from 1.5 to 5 times the pressure measured in drained dams and from 1.5 to 2 times that found in dams that are not drained.

EFFECTIVE UPLIFT AREA

The uplift force, with which the designer of dams is principally concerned, is the product of pressure and area. The latter factor may be of even greater importance than the former, but the determination of the area over which uplift pressure is effective is far more difficult than the measurement of pressure.

As regards pore-water uplift a fairly accurate estimate of area is possible. The pore space and, consequently, the pore area, in good concrete cut by any plane will probably occupy from 10 to 12% of space and area, respectively.

Effective uplift area—in joints, in incipient cracks, along the junction of rock and concrete, or in seams in the rock just below the base—is a factor that has perplexed designers because of the complete absence of known facts and because of the variability of local conditions.

In an effort to throw some light on this subject as regards concrete construction joints, tests were conducted at the laboratory of the Bull Run Dam in 1927. The results were negative in that they indicated that bond between concrete and concrete, as made in the laboratory (even in careless work), is practically as tight as the concrete above and below the bonded plane. Similar results were obtained[9] two years later in Pasedena, Calif., by the late John Egon Skafte, Jun. Am. Soc. C. E., under the supervision of Franklin Thomas, M. Am. Soc. C. E.

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[8] *Civil Engineering*, Vol. 2, No. 9, September, 1932, p. 580.

The fact that occasionally water under pressure has been found in field construction joints and that seepage water has been frequently observed to issue at the intersection of such joints with the down-stream face justifies the conclusion that water movement may be due to imperfections in mass concrete regarding which laboratory work can give no information other than that, with care, a tight bond can be secured.

Nothing is known of effective uplift area at the base of a dam where concrete comes in contact with rock. All that research has revealed thus far is that, at the great majority of points at which measurements were attempted, both pressure and measurable flow were found. From this it may be inferred that the relative effective area is greater at the base than in concrete construction joints. This conclusion seems reasonable when the physical differences are considered between the two materials in contact. These refer in particular to the elastic moduli and the coefficients of expansion by heat, both of which are certain to set up severe local stresses which may result in separation. Furthermore, the foundation may contain deep-seated seams that may admit water to the base around any cut-off or grouting screen that is provided. It has not yet been possible to appraise these factors which make the problem a practical rather than a theoretical one.

There is one factor, however, that must have considerable influence on potential uplift area, namely, intensity of loading. It is evident that heavy compression at contact faces must greatly hinder, or even stop, all percolation except through pore areas. Inversely, in the case of light loading, the uplift force in areas of imperfect bond and in pores may put the concrete in tension.

If H is the depth, in feet, below water of a point on the water face, and therefore, likewise expresses the maximum intensity of uplift pressure per square foot (62.5-lb unit) near the water face, and if e is the proportion of uplift pressure area to gross area around that point, then eH is the unit uplift force at that point.

It is conceivable that with no loading and with perfect adhesion, separation would not necessarily follow. However, dependence cannot be placed on tensile strength at concrete construction joints and especially at bond between concrete and foundation rock. When eH is greater than unit loading, tension will exist. Should a tension break occur, the uplift pressure area will increase and the resultant uplift force, acting as a wedge, will tend to cause progressive separation and to an increase of pressure area. It is of great importance that this be avoided, and to that end surplus vertical loading must be provided at the water face.

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[9] Unpublished thesis, California Inst. of Technology, Pasadena, Calif., 1929.

Many small dams have been built with a vertical water face and a downstream slope of 0.70, which corresponds to a heel loading (reservoir full), of about 0.36 *H*. As, in general, no progressive increase of percolation or uplift pressure has been noted it may be argued that the effective uplift area at the water face did not exceed 36 or, say, 40 per cent. This percentage equals more than three times the pore area and is suggested as a fair and safe estimate under such loading conditions, taking careful field work for granted.

For lighter loading it may be supposed that this percentage increases rapidly. On the other hand, with heavier loading, the potential percentage will probably decrease and the somewhat arbitrary relation listed in Table 6 (Columns (2) to (5)) may be reasonably considered, on the assumption that the dam is founded on good rock.

TABLE 6.—ASSUMPTIONS AS TO UPLIFT

Vertical loading at the heel	Effective Uplift Pressure Area (Percentage of Total)				Average uplift pressure, in 62.5-lb. units	Uplift force, *U*, in 62.5-lb. units
	At heel	At toe	Average	Relative area subject to uplift (proposed for design)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.4 *H*	40†	25	32.5	40	0.5 *H*	0.200 *HL*
0.6 *H*	35	25	30.0	35	0.5 *H*	0.175 *HL*
0.8 *H*	30	25	27.5	30	0.5 *H*	0.150 *HL*
1.0 *H**	25‡	25	25.0	25	0.5 *H*	0.125 *HL*

*Allowance for pore-water pressure. † Three times pore area. ‡ Twice the pore area.

It would be logical when considering the entire base to use the percentages in Column (4) of Table 6. It must be remembered, however, that it is mostly imperfections in concrete and rock base that govern these percentages and that an accurate appraisal of these imperfections is practically impossible. The more conservative course is recommended, therefore, namely, to use the values in Column (5) for the entire base.

UPLIFT FORCE

The estimates for design made first of uplift pressure and next of uplift area may be combined in order to arrive at the uplift force (see Table 6, Columns (6) and (7)). In Column (7), Table 6, *L* is the length of base, in feet.

The existence of an uplift force due to water under pressure in the pores of concrete has been questioned. This argument has been definitely settled, however, at least in the case of mortar, by unpublished experiments made by Mr. Woodard. Mr. Woodard tested the middle part (2 in. high), of the cylindrical surface of 2 by 4-in. mortar specimens of various mixtures and subjected them to water pressures up to 2 000 lb per sq in. in an apparatus that permitted the ends to be left free and exposed to the air. The water entering the pores percolated slowly toward the outer ends, and in the interior it exerted pressure which, acting in the pore spaces, produced an axial force sufficient to break the specimen.

The breaking force was estimated from the hydraulic pressure necessary to cause the break and the ascertained porosity. The tensile strength of the specimens was found by breaking control briquettes. Column (7), Table 7, shows the relation between the force necessary to break the specimen and the force computed to be required from the tensile strength.

That the results are far from consistent may be partly explained by the fact that the briquettes may not have furnished correct values of the tensile strength of the cylinders. Furthermore, the water pressure in the pores of any normal section cannot have been uniform throughout the full area; it was undoubtedly smaller at the center than at the circumference. The average may have easily been a different proportion of the measured outer pressure for the various mixtures. The important fact has been demonstrated, however, that water in pores can exert an uplift force.

TABLE 7.—WOODARD TESTS TO DETERMINE FORCE OF WATER IN THE PORES OF MORTAR

Number of specimens broken (1)	Mixture (2)	AVERAGE VALUE				
		Hydraulic breaking pressure, in pounds per square inch (3)	Porosity (percentages) (4)	Breaking force, in pounds per square inch (5)	Strength of briquettes, in pounds per square inch (6)	Breaking force / Strength of briquettes (7)
18........	Neat	1 411	31.0	438	477	0.92
11........	1:1	1 485	25.9	384	454	0.85
24........	1:2	1 514	20.3	305	475	0.64
22........	1:3	959	17.1	164	437	0.37
10........	1:4	191	19.3	37	349	0.11
10........	1:5	165	25.3	42	240	0.17
12........	1:6	122	29.5	36	173	0.21

Accepting, as a fact, the possible existence of uplift force due to the pressure of water in the pores of mortar, there is every reason to conclude that a similar force may be exerted in the body of concrete even if its potential amount will probably be less because of smaller pore space.

The average pressure in pores may be considered as governed by the same law as that in fine seams. Therefore, a value of $0.5H$ was assumed in Columns (6) and (7), Table 6. This table, in Column (7), contains recommended safe estimates of uplift force in dams in which the vertical loading at the up-stream face varies from H to $0.4H$. At this lower limit of loading (reservoir full) a balance may be estimated to exist between local uplift force and vertical loading at the water face. In dams with still lighter loading this may be exceeded by uplift force, and tension may develop in the concrete. As was argued under the heading "Effective Uplift Area," the danger of this condition lies in a possible increase of uplift area, resulting, of course, in proportionate force increase.

EFFECT OF UPLIFT FORCE ON STABILITY

In studying principles governing the effect of uplift on stability it is convenient to consider the unit of an ideal dam of infinite length with triangular profile having a vertical water face, a horizontal base, and water at the apex level, as shown in Fig. 5(*a*).

Water pressure in the pores of concrete implies permeability. Concrete must be workable when it is placed and usually more water is required for this purpose than is necessary for completing chemical action. Therefore, the interior of massive concrete may be expected to have its pores filled with water if air has been excluded in placing and if no loss has been suffered from surface evaporation.

Excavation and core drilling in dam concrete have shown the mass to remain fully saturated for an indefinite time even without water against the dam except that an outer shell will have partly dried. The depth of this shell is relatively small in a large dam.

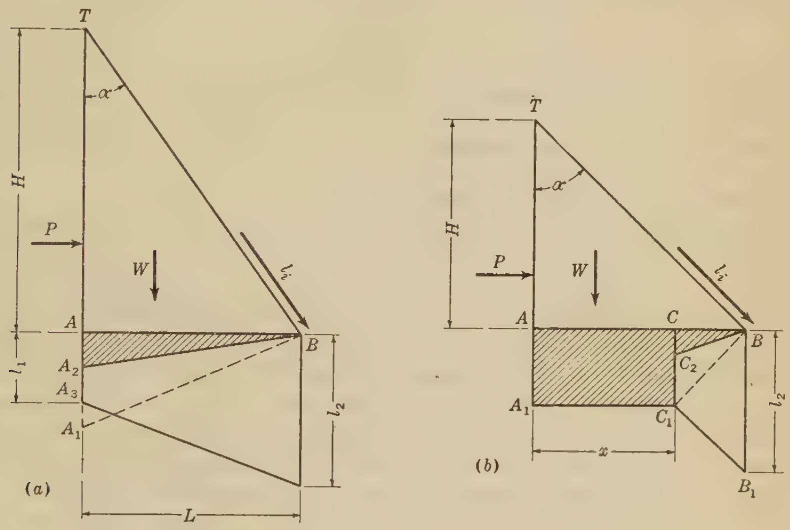

FIG. 5.

Concrete having 12% pore space, and with an average specific weight of aggregates varying from 2.65 to 2.60, will have a mass specific weight, oven dry, ranging from 2.34 to 2.29, and when fully saturated from 2.49 to 2.44. The weight of pore water above any sliding plane and the pore-area uplift on such plane will tend to balance each other. Since pore uplift will be considered separately the specific weight of mass concrete must be computed on the basis of concrete that is mostly saturated and will be taken at 2.4 in this study.

The potential effect of heel loading on uplift pressure area and force has been discussed. In order to estimate uplift force, the toe loading must be known. In this paper it is proposed to determine the toe loading on the assumption of straight-line distribution of normal foundation stresses. This assumption does not accord with the elastic theory, but is made because the great uncertainties as to uplift forces do not justify great refinement in a general treatment of the subject. It should be understood, however, that in important individual cases such refinement may be desirable and important.

It can be shown that, on the assumption of a linear distribution of normal stresses and no effect on loading of uplift (see Fig. 5(*a*)),

$$l_1 = H\left(a - \frac{1}{n^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$l_2 = \frac{H}{n^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

and,

$$l_4 = H + \frac{H}{n^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which, H = height of dam, in feet; L = width of base of dam, in feet; $n = \tan \alpha = \frac{L}{H}$; α = angle between the up-stream and the down-stream faces of the dam; and, a = specific weight of concrete.

The uplift pressure may be represented, as previously stated, by a straight line, such as $A_1 B$ in Fig. 5(*a*). The uplift force may be correspondingly represented by the line, $A_2\,B$, if AA_2 bears a proper relation to AA_1.

So long as AA_2 is less than AA_3 (the heel loading), no alteration of loading occurs. If, however, the summit angle is so small that l_1 is less than the limit previously estimated at $0.40H$, there is danger that uplift force will cause progressive separation. This will be accompanied by increase of pressure and pressure area, which may extend to a point of the base where the intensified vertical loading, due to load shift, equals full reservoir pressure.

Thus, a condition may be created, such as is illustrated in Fig 5(*b*), in which the entire area, $AA_1\,C_1\,B_1\,B\,A$, represents the weight of the dam; and AC is the length of dam base over which gradual separation due to uplift has taken place, and for which the uplift pressure may be equal to the full water height effective over 100% of the area. The loading equals full water pressure, H, at Point C, and the triangle, CC_2B, represents pore-area uplift force.

For such a case (see Fig. 5(*b*)), assuming the specific weight of concrete to be 2.4:

$$x = \frac{2.5 - 3.5\,n^2}{n}\,H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$l_2 = \frac{4.9\,n^2 - 2.5}{4.5\,n^2 - 2.5}\,H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and,

$$l_4 = \frac{(4.9\,n^2 - 2.5)\,(1 + n^2)}{4.5\,n^2 - 2.5}\,H \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

From Equation (10) it will be seen that $x = 0$ for $2.5 - 3.5n^2 = 0$, from which, $n = 0.843$, which is the condition proposed by Levy; that is, loading at the water face must not be less than the water pressure at that elevation.

It will also be noted from Equation (11) that the vertical toe loading, l_3, will become ∞ for $4.5n^2 = 2.5$, or $n = 0.743$.

Thus, it would appear that when the rock foundation is so bad that unusually great uplift pressure should be expected, a value of n greater than 0.743 must be used to prevent crushing at the toe. This would not mean, however, that a value of n of 0.843 would be necessary under all conditions, as advocated by Levy. Even with a lower value of n there may yet be ample resistance along the portion, BC, of the base to give the required security against down-stream movement.

Under ordinary conditions of reasonably tight foundation and on the basis of assumptions previously made, the relation in the ideal dam of dimensions and uplift force is shown in Table 8, Items Nos. 1 to 6. In order to evaluate the probable effect of uplift on stability, Equation (6) may be used, in which it becomes necessary to assign values to s_1 (unit shearing strength under no load) and k (factor of shearing strength increase due to load).

TABLE 8.—RELATION BETWEEN DIMENSIONS OF AN IDEAL DAM, UPLIFT FORCE, SHEAR SAFETY FACTORS, AND NOMINAL SLIDING FACTORS

Item No. (1)	Description (2)	Heel Loading, Reservoir Full						
		0.4 H (3)	0.5 H (4)	0.6 H (5)	0.7 H (6)	0.8 H (7)	0.9 H (8)	1.0 H (9)
1	Ratio, $\frac{L}{H} = n$	0.707	0.725	0.745	0.766	0.790	0.815	0.843
2	Percentage of base area subject to uplift pressure...........	40.0	37.5	35.0	32.5	30.0	27.5	25.0
3	Average uplift pressure	0.5 H	0.5 H	0.5 H	0.5 H	0.5 H	0.5 H	0.5 H
4	Uplift force, U.......	0.141H^2	0.136H^2	0.130H^2	0.124H^2	0.118H^2	0.112H^2	0.106H^2
5	Weight, W, of a linear foot section of dam	0.848H^2	0.870H^2	0.894H^2	0.910H^2	0.948H^2	0.978H^2	1.013H^2
6	Difference, $W - U$...	0.707H^2	0.734H^2	0.763H^2	0.785H^2	0.829H^2	0.865H^2	0.907H^2
7	$1.4 \frac{W-U}{H^2}$	0.99	1.03	1.07	1.11	1.16	1.21	1.27
8	$1.4 \frac{W}{H^2}$	1.19	1.22	1.25	1.28	1.33	1.37	1.42
9	Increase in factor of safety, Q, if uplift is eliminated.........	0.20	0.19	0.18	0.17	0.17	0.16	0.15
	Factor of safety, Q, in Shear:							
10	$H = 200$..........	7.49	7.71	7.92	8.16	8.43	8.71	9.03
11	$H = 300$..........	5.32	5.48	5.64	5.81	6.00	6.21	6.44
12	$H = 400$..........	4.24	4.37	4.50	4.64	4.79	4.96	5.15
13	$H = 500$..........	3.59	3.71	3.81	3.93	4.07	4.21	4.37
14	$H = 600$..........	3.11	3.25	3.35	3.46	3.58	3.71	3.86
15	$H = 700$..........	2.85	2.93	3.01	3.11	3.23	3.35	3.49
16	Safety factor against nominal sliding, on the basis of a friction coefficient of 0.75..	1.06	1.10	1.15	1.19	1.24	1.30	1.36

Until the results of additional research become available the experiments herein quoted, so far as they go, justify a value of 0.70 for k and of 800 lb per sq in. for s_1. For sound rock the value of s_1 is much higher, but in mass formation it is subject to far greater uncertainty. In view, also, of uneven distribution of shear (the maximum being generally less than twice the average), and the possibility that some areas of construction joints in the dam, although staggered or stepped, may coincide with a potential plane of shear movement—it is proposed in this study to give s_1 a value of 400 lb per sq in. In the units of previous equations this is equivalent to $400 \frac{144}{62.5} = 920$.

Introducing these values in Equation (6) and considering that the base area is nH and the horizontal water pressure, $P = \frac{1}{2}H^2$:

$$Q = \frac{1\,840\ n}{H} + 1.4\,\frac{W - U}{H^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots(13)$$

In order to indicate the relative effect of uplift, the values of the second term of Equation (13) are shown in Items Nos. 7 and 8 of Table 8, in cases of uplift and no uplift.

Item No. 9 of Table 8 shows the numerical decrease in Q, due to uplift. Its relative importance can be judged by comparison with values of Q, found by Equation (13) and listed as Items Nos. 10 to 15, in Table 8. Comparing these items, involving uplift, with Item No. 9, it is found that in cases where Q equals 4 or more the factor of safety increase resulting from eliminating uplift, ranges from 3% to 5%, assuming that the water-face loading, with reservoir full, does not drop below $0.4H$, and that a reasonably tight foundation exists.

Where the foundation is so seamy and permeable as to permit conditions to arise, as illustrated in Fig. 5(*b*), a far more serious effect should be anticipated although, even then, safety may be obtained by increasing the dimensions.

The problem requires special treatment when the foundation contains extensive horizontal stratifications. In such a case, movement on or in the foundation may take place without overcoming shear resistance and may merely encounter frictional resistance, becoming thereby equivalent to the cases of loose rock-fill and earth dams. Then, uplift may have its most serious, and easily its most dangerous, effect on stability unless dam dimensions have been greatly increased. The mere presence of water in horizontal foundation seams tends to reduce the coefficient of friction; and, in addition, with water under pressure the friction-producing load may be greatly reduced. This was well illustrated in the failure of the dam at Austin, Pa.[10]

Caution in designing a dam will also be particularly important when the rock foundation is of a nature such as will cause the material to soften in the long-continued presence of water. After slow and almost unavoidable water percolation, especially where no deep cut-off is provided and base drainage is neglected, settlement must be expected, with dams of usual dimensions, at the down-stream toe. This settlement is likely to be followed quickly by freer access of water under the up-stream toe, which will result in nearly complete destruction of the shearing resistance of the base. At the same time, through uplift, it will result in a large reduction of frictional load and its resultant friction, which is all that remains to oppose the downstream movement. It is believed that the failure of the St. Francis Dam started in that manner.

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[10] *Engineering Record*, Vol. 7, 1911, p. 429.

EFFECT OF HEIGHT ON BASE LENGTH

Table 8 (Items Nos. 10 to 15) shows in the ordinary case of good rock foundation, the factor, Q, for dams of various heights and indicates that, comparing dams of the same proportions, the higher the dam, the less safe it is. This fact may be also directly inferred from Equation (13) in which the second factor, $1.4\,\frac{W-U}{H^2}$, is for dams of common section, practically, and for dams of triangular section, entirely independent of H, since W and H both vary with H^2, and H cancels out. The first factor, $\frac{1\,840\,n}{H}$, is decidedly the major item, and this factor becomes smaller with increase of height.

The economic profile of a dam with vertical face, considering only the element of down-stream mass movement, may be readily determined for any factor of safety assuming shearing strength to be known. If, as has been suggested, $0.4H$ is taken as the lowest permissible face loading, a value of $n = 0.707$ follows for the upper part of the dam where, with such value, shearing strength is excessive. Introducing this value in Equation (13): $Q = \frac{1\,301}{H} + 1.4\,\frac{W-U}{H^2}$; and since $W = 1.2\,H^2 \times 0.707$ and $U = 0.2\,H^2 \times 0.707$; $Q = \frac{1\,301}{H} + 0.99$. For $Q = 4$, the head, H, is found to be 432 and the base length, L, becomes 305.4, all linear dimensions being in feet.

For heights greater than 432 ft, Q will become smaller than 4 unless the down-stream slope is flattened (see Fig. 6). By the further use of Equation (13), it will be found that for $H = 500$, 600, and 700 ft, respectively, the required base lengths are 401, 565, and 742 ft, all on the following assumptions pertaining to a vertical-face profile (see Fig. 6): Specific weight, a, of concrete, 2.4; minimum loading on the water face, $0.4H$; uplift force, as given in Table 6, Columns (6) and (7); shearing strength, $400 + 0.70q$; and minimum factor of safety, Q, in shear, 4.

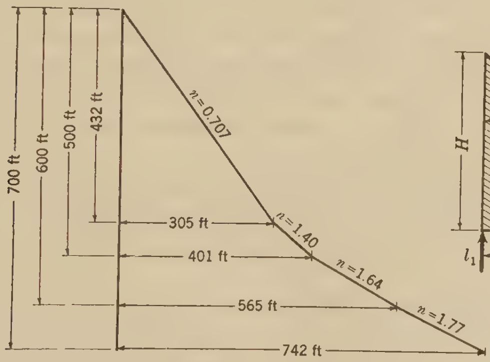

FIG. 6.

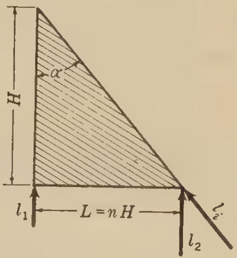

FIG. 7.

Safety limitation of the stresses in the concrete has been disregarded for the sake of simplicity, but it can be easily introduced into the problem. For instance, if 600 lb per sq in. is determined as the maximum compressive stress, this limit with reservoir empty would be reached, in a dam with a vertical up-stream face, at a height of 575 ft. Below this elevation it would be necessary to introduce a slope to the water face which, in turn, would cause a change in the down-stream slope.

The important fact to which attention is directed is that the maintenance of a definite safety factor in shear, for high dams, will require a rapidly increasing relative base length.

SHEAR SAFETY FACTOR *Versus* SLIDING FACTOR

The general use that has been made in recent years of the sliding factor in judging the sufficiency of a dam profile, may have originated in the consideration that masonry, and especially concrete, dams are often constructed in horizontal courses between which adherence may be imperfect and such use may have been favored by the simplicity of the method and possibly also by the relative lack of knowledge of shearing strength.

The continued use of the sliding factor is incomprehensible nevertheless now that the necessity of securing a maximum of shear resistance by roughening and cleaning the base and by stepping or sloping the construction joints is so thoroughly understood and practiced, simple sliding is out of the question.

The only logical use of the sliding factor may be found in case the aforementioned essentials are disregarded, or where the foundation rock is made up of extensive horizontal, or almost horizontal, strata.

In ordinary cases of a reasonably sound foundation free from such strata the use of the shear safety factor is logical. It brings to mind the necessity of designing and constructing the dam so as to develop maximum shear resistance and makes due allowance for the effect of height on safety.

The use of the shear safety factor may be objected to because the distribution of shearing strength and shearing stresses may not coincide. Thus, in spite of apparent safety as to mass movement, the local stress may exceed the local strength, and progressive shear failure may result. In ordinary cases the maximum shear stress will always be less than twice the average, and this range is fully covered by such assumptions as were made in this study.

On the whole, it appears that all arguments point to the logic in dam design of using the shear safety factor and of abandoning the sliding factor, except only where sliding may take place on horizontal construction joints or foundation strata without involving shear. Only in such case can the use of the sliding factor be considered logical and, if so used, a suitable safety factor should be introduced.

UNUSUAL UPLIFT CONDITIONS

At some dam sites especially severe conditions may occur, such as possibilities of vibration due to heavy overflow, to earthquakes, or to badly seamed foundation, which may render the estimate of uplift pressure area in Table 6 unsafe. In such cases recourse may be had to studying the worst uplift conditions readily conceivable as illustrated in Fig. 5(*b*).

Shear value as well as frictional resistance for the base length, AC (see Fig. 5(*b*)) must be considered destroyed, all resistance to motion being confined to the remaining contact along the base length, CB. On the basis of unit values previously used, shear resistance is,

$$920\ (Hk - x) + 0.70\ (1.2Hn - U)$$

The value of uplift up stream from C is Hx, and down stream from C, where it results from pressure in pore area only, it is $0.125H\ (Hn - x)$, hence,

$$U = Hx + 0.125\ (Hn - x)H \quad \ldots\ldots\ldots\ldots(14)$$

Introducing for x the value given in Equation (10):

$$U = 0.125\ H^2\ \frac{17.5 - 23.5\,n^2}{n} \quad \ldots\ldots\ldots\ldots(15)$$

A factor of safety of 4 against movement is obtained by compliance with the equation:

$$4P = S + 0.7\ (W - U) \quad \ldots\ldots\ldots\ldots(16)$$

The final safety requirement may be that the inclined toe loading (Fig. 7), shall not exceed a certain limit, say, 600 lb per sq in., which leads from Equation (12) to: $600 \times \dfrac{144}{62.5} = \dfrac{(4.9\,n^2 - 2.5)\ (1 + n^2)}{4.5\,n^2 - 2.5}\,H.$

The different values found from these equations are plotted in Fig. 8 as is also the Levy requirement which, by Equation (10), gives a value to n of 0.843. It will be noted that control is had for dams of less height than 575 ft by Curve Q (Fig. 8), which automatically provides for the inclined stress limitation. The spread between Curve Q and the line representing the straight-line Levy requirement is emphasizd in Fig. 8 by shading, and indicates its unnecessary excess.

NUMERICAL EXAMPLES

It may be useful to present some numerical values of the sliding and the shear safety factors for cross-sections of a few existing dams. This is done in Table 9 on the basis of considering a linear foot of dam by itself and assuming that it can move down stream by overcoming base shear only. The same shearing strength values are used as heretofore, although careful investigation would undoubtedly have assigned different values to each individual case. Table 9 makes it additionally apparent that the shear safety factor furnishes a more rational criterion by which to judge the safety of dams than the sliding factor.

GENERAL REMARKS

It should be emphasized that while the shearing strength of concrete may be estimated on a reasonable basis of experiment there remains a great uncertainty as to the mass shearing strength of foundation rock. It may vary materially from place to place, and especially between river valley and abutment sections. This uncertainty will naturally lead to the adoption of a very conservative value, such as was used in this study. Even this may be far too high in the case of poor foundation rock.

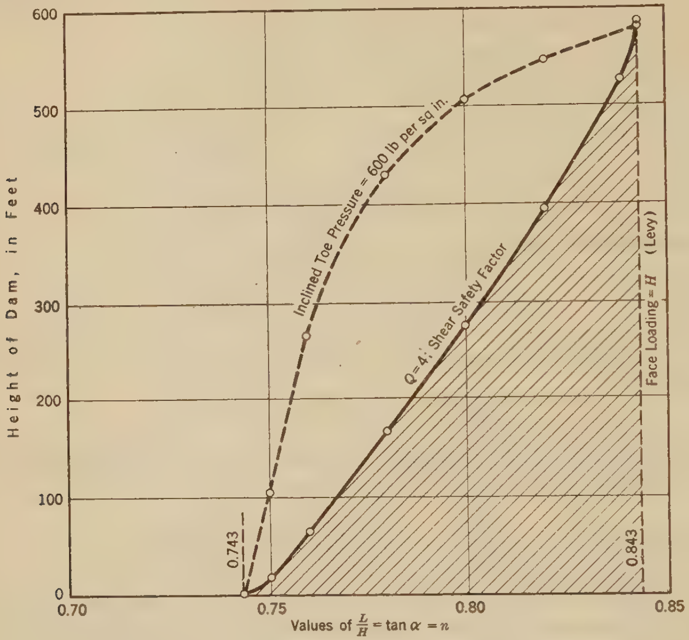

FIG. 8.—SAFETY LIMITATIONS; TRIANGULAR DAM SECTION

The shear safety factor may be determined easily for any linear foot of a dam if shear resistance on the base is the only item taken into account. However, allowance must be made for shear resistance which may develop

TABLE 9.—FACTORS OF SAFETY AGAINST NOMINAL SLIDING AND SHEAR

Description	American Falls	Bull Run (considered straight)	Elephant Butte
Height, in feet	79.5 below high water	200	254
Base width, in feet	60	148.5	205.6
Slope of water face	Vertical	25 on 1	16 on 1
Dry slope	3 on 2	7 on 10 and 8 on 10	2 on 3 and 1 on 1
Horizontal force *	3 052	19 950	30 600
Weight on foundation, W*	5 970	36 900	62 650
Heel loading *	$0.44H$	$0.60H$	$0.69H$
Ratio of effective uplift area to gross area	39%	35%	33%
Uplift force, U*	930	5 470	10 680
$W - U$*	5 040	31 430	51 970
$0.70\ (W - U)$*	3 530	22 000	36 400
Base shear (no load), basis, 400 lb. per sq. in.*	55 200	136 400	189 100
Total base shear*	58 700	158 400	225 500
Ordinary sliding factor	0.61	0.63	0.59
Apparent factor of safety (basis, sliding factor, coefficient of friction, 0.75)	1.23	1.19	1.27
Shear safety factor, Q	19.1	7.9	7.3

* 62.5-lb. units.

at the ends of the section considered. The safety factor may also be determined for the dam as a whole not only in the instance of a straight gravity dam, but also for curved gravity, gravity arch, and arch dams. In such cases a trough-like surface may be assumed closely approximating concrete and rock contact along which shear movement would most likely take place. Such a general surface would have horizontal contours parallel to the direction of the supposed mass movement, or circular contours if rotary motion is considered about some vertical axis in or near the end of the dam.

Cantilever and arch stresses along the surface of such a trough will produce resultants, the aggregate horizontal components of which in the direction of motion, make up the total shearing force, which should equal the water pressure on a projected plane normal to the direction of motion. The aggregate numerical value of all components normal to the trough surface, less the corresponding uplift force, furnishes the load that causes an increase in shear, while the total area of the trough, less that part, over which adherence may be destroyed by uplift or tension from other causes, may be taken as the source of shear resistance exclusive of the increase due to load.

The rules of uplift determination would be the same as those suggested in connection with straight gravity dams, the principle being identical in all cases.

Acknowledgment

Acknowledgment is due to Silas H. Woodard, M. Am. Soc. C. E., for permission to use the valuable results of his research relating to mortar shearing strength and stresses resulting from pore water under pressure in mortar cylinders.

DISCUSSION

H. DE B. PARSONS,[11] M. AM. SOC. C. E. (by letter).—The writer agrees with Mr. Henny that shear is a more important factor than sliding, and that consideration of a safety factor should be based more on the former rather than on the latter. With good, hard rock, properly prepared and roughened and with all loose parts removed, it is difficult to conceive that true sliding could occur. Under such conditions, sliding at the foundation plane could take place only by shearing some of the rock, and the shearing strength of good rock exceeds that of concrete. With a rock that is likely to soften under continuous water pressure, the danger is generally greater at the side abutment joints than under the central base of the dam. Security against such weakness is best provided by a suitable design.

If possible, the surface of a rock foundation should be graded upward in the down-stream direction. Such grading gives a dam additional resistance to movement down stream, and, also, permits a component of any uplift pressure to act up stream against the hydraulic pressure.[12]

On a pervious soil foundation, there is danger that a masonry dam may slide, although, with a properly designed base, any sliding of the structure would carry some of the foundation material with it; that is, any sliding would cause shearing of the soil. For such a foundation, the writer advocates a wide base, and such a sectional profile of the dam structure that the reservoir water will add weight to hold the dam in position. This can be done by designing the up-stream face on a rather flat slope.

The intensity per unit of area of the uplift pressure on the base of a dam may be greater with some classes of good rock foundation than with broken or fissured rock or pervious soil. Under the latter conditions the hydrostatic head may be lessened since the water has a freer exit from under the base. Nearly all recorded uplift intensities are less than that represented by an even gradient from reservoir surface to tail-water;[13] therefore, Mr. Henny's proposition to estimate the average intensity at 0.50 H should be safe. In designing dams, especially those that are high and important, it is well to make assumptions that tend toward safety.

The effective area of the base on which this intensity of pressure acts cannot be predicted with absolute certainty in the case of rock foundations As the rocks present many variable conditions, it is only by repeated investigations and the increased knowledge thus gained, that effective area may be approximated in percentage of total area, so that fair accuracy can be obtained.

The uplift pressure will not be uniformly distributed. If the effective area on which the uplift acts is, say, 35% of the total, the effective part of any selected part of the total may be more or less than this percentage.

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[11] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

[12] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 185.

[13] "Hydrostatic Uplift Under Masonry Dams," by H. de B. Parsons, M. Am. Soc. C. E., *Proceedings*, World Eng. Congress, Tokyo, 1929, Vol. IX, p. 295.

Grouting under the up-stream part of the base of a dam will not remove entirely the hydrostatic uplift from under the structure, no matter how well it may be accomplished. Uplift has been found under dams that have had the foundation rock well grouted.

When the foundation is a pervious soil, uplift can be expected to act on the full area of the base with an intensity diminishing from the up-stream corner to the down-stream corner. While the uplift at the down-stream corner will correspond to the tail-water head, the uplift at the up-stream corner will be considerably less than that due to reservoir head, depending on the efficiency of the cut-off. There should not be a cut-off or obstruction of any kind at the down-stream part of the base that would prevent the water from having a free exit. If there is such obstruction the hydrostatic uplift will be increased.

Mr. Henny has brought out very clearly the necessity for making an allowance for uplift force in the design of a dam, on either rock or pervious soil. For important dams, this allowance should be generous rather than meager. Considerable information of actual uplift intensities under dams is now available. The writer believes that foundation materials could be classified and that to each could be assigned a safe assumption of uplift intensity. While the lower limit would be the tail-water head at the down-stream corner of the base if the tail-water surface was above the base, and zero if the tail-water was below it, the maximum limit, taken at the up-stream corner of the base, would vary from a fraction of the reservoir head above the base to full reservoir head. Thus, for a specific class of foundation material, the maximum intensity could be taken at some figure between $f\ H$ and H, diminishing uniformly to tail-water head if above the base, or to the elevation of the down-stream corner if below. In like manner, the percentages of effective areas of the base could be classified with reference to the character of the foundation material. Such assumptions, for present purposes of design, could be corrected to closer classifications from time to time by additional observations and researches, and increased knowledge obtained therefrom.

The writer avoids the expressions "heel" and "toe." Dams are now designed with such a variety of shapes, that these words have become meaningless as they have lost their pictorial value. Uniform terminology should be encouraged, and better expressions are "up-stream corner" or "down-stream corner," which leave no uncertainty as to which corner or edge of the base is meant. Similarly, it is customary to refer to "length" as the axial measurement of a dam from abutment to abutment, although Mr. Henny has used the word as meaning the "width" of the base. Engineers should set an example in the use of correct expressions when writing on technical subjects.

A. A. EREMIN,[14] ASSOC. M. AM. SOC. C. E. (by letter).—In this paper the author has developed an interesting criterion of stability of the straight gravity dam with consideration of shear resistance. Mr. Henny has pointed out correctly that shear strength in masonry is difficult to determine. The angle of the plane of failure in compression test specimens varies with the physical qualities of the material and the method of transmitting compression forces. Professor A. Morley[15] has shown the failures in rock specimens subjected to direct compression in which the planes of failure are almost in the same direction as that of the compression forces, indicating, therefore, that the specimen has failed from tensile stresses. The compression forces were applied to the specimen through thin steel plates. Similarly, tensile failure occurs in compression tests if steel plates are covered with paraffin.[16] It is almost impossible to obtain a uniform distribution of compressive stresses on the plates and the initial failure usually occurs from tensile stresses.

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[14] Associate Bridge Designing Engr., State Highway Comm., Sacramento, Calif.

In Coulomb's shear equation cited by Mr. Henny the elastic properties of material are not considered. Therefore, it may be applied for the determination of shear resistance in brittle materials only. Professor O. Mohr has shown that shear strength determines the elastic properties of material and also that a plane of the maximum shear stresses is equi-distant from the maximum and minimum normal stresses. In practice, a distribution of the maximum shear stresses has been proved by Lüder's lines. At present, valuable facts concerning stress distribution are being revealed by photoelastic analysis based on the distribution of maximum shear stresses in the elastic materials.

Measurements have indicated the pronounced deformations in straight gravity dams.[17] It remains to be determined whether these deformations are due to slippage or elastic deformations in concrete and foundation material. With sufficient experimental data the elastic method will yield the most economic and safest design. However, the middle third theory, the sliding factor theory, and Coulomb's internal friction theory are convenient guides to the formation of judgment as to the stability of structures.

CALVIN V. DAVIS,[18] M. AM. SOC. C. E. (by letter).—It is gratifying to note that Mr. Henny has shown the fallacy of considering the sliding factor as a measure of the safety of a gravity dam against down-stream movement. His substitution of a factor of safety against shearing failure for the sliding factor is entirely logical and is in keeping with modern design methods.

The author concluded from certain cylinder tests conducted by the U. S. Bureau of Reclamation "that there is a persistent increase of shearing strength with increasing load." This conclusion, which agrees substantially with Coulomb's theory, is of great importance in the design of both gravity and buttress dams, but applies with particular force to the latter.

These tests and the laws derived therefrom show that in the design of buttress dams the factor of safety against shearing failure would be increased if the second principal stresses have substantial values in compression throughout the buttresses. This is particularly true near the up-stream buttress faces where the critical stresses most frequently occur. This principle has been ignored all too frequently in the design of buttress dams.

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[15] "Strength of Materials," by A. Morley, 1917, p. 524.

[16] "Strength of Materials," by Prof. S. P. Timoshenko, Russian Edition, p. 28.

[17] *Civil Engineering,* August, 1932, p. 489.

[18] Chf. Designer, Ambursen Dam Co., Inc., New York, N. Y.

The importance of the intensity and distribution of shearing stresses in a buttress dam was also recognized by Dr. A. Stucky,[19] a Swiss engineer, who made a careful study of the causes of failure of the Gleno Dam in Italy. Dr. Stucky concluded that the shearing failure of the buttresses of this multiple-arch dam was a contributing cause of this disaster. While there was considerable controversy over Dr. Stucky's conclusions, the principles of design introduced in his report are of importance and are intimately related to those developed by Mr. Henny.

An important relation exists between shearing stress intensity and distribution and the structural safety of dams. In using modern designing methods in the proportioning of buttress dams the writer has experienced no difficulties in obtaining a substantially uniform distribution of the first principal stresses, the vertical normal stresses, and the shearing stresses on any horizontal plane. Such stress distribution gives both maximum economy and maximum safety. If a buttress dam is proportioned for uniform shearing stresses on any horizontal plane, Equation (2) is correct for a value of S equal to the actual shearing strength of the foundation material. No reduction factor such as those which Mr. Henny has included in his equations will be necessary to take care of the high localization of shearing stresses such as occur at the down-stream edge of a gravity dam.

The trend toward the use of inclined joints in both gravity and buttress dams makes consideration of the shearing stresses all the more important. If such joints are used the factor of safety against shearing failure of the down-stream column formed by the joint should be computed independently of the remainder of the structure. In the straight gravity type of dam the highest shearing stress intensity would occur in this column, and its factor of safety would be much less than that of the dam as a whole.

In regard to sliding factors, the writer's experience has been that if dams of both the gravity and buttress type are conservatively designed in respect to the intensity and distribution of the principal and shearing stresses, the sliding factor will have a value that is well within accepted limits. For example, in a recently completed design, for a buttress dam, 400 ft high, the sliding factors decreased from a value of 0.75 at a height of 200 ft to 0.64 at a height of 400 ft. The maximum first principal stress in this design was maintained at a uniform value of 450 lb per sq in. throughout the entire structure, and the second principal stress had substantial values in compression at the critical points. The writer has been called upon frequently to analyze designs for dams having sliding factors much higher than those allowed by accepted practice. In all cases, an investigation of these designs revealed that the first principal stresses were far from conservative, the second principal stresses had high values in tension, and the shearing stresses were excessive.

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[19] *Engineering News-Record,* March 20, 1924, p. 486.

The author is correct in stating that drains in a gravity dam can not be depended upon to eliminate uplift. In view of the great base length required to provide safety against both uplift and shear for high dams, as demonstrated by Mr. Henny, the writer believes that economy will require the use of modified types of gravity dams in which uplift is positively eliminated. A number of such types were developed by the late Fred A. Noetzli, M. Am. Soc. C. E.[20] These consisted of gravity dams containing sufficient cellular spaces to provide complete drainage. These cellular spaces are of such size that clogging would be impossible.

William P. Creager,[21] M. Am. Soc. C. E. (by letter).—This ably presented paper deals with a subject that is very important to the designers of high, solid, gravity dams. Included in Table 8 is a mathematical demonstration of the fact, frequently expounded by the writer,[22] that the factor of safety against sliding is much less for high dams than for low ones.

This feature is more impressive when shown diagrammatically. In Fig. 9, the writer has plotted the relation of friction and shear to the factor of safety as applied to Column (3) in Table 8. The decreasing effect of shear for high dams is very clear; it contributes relatively seven times as much to the safety of a 100-ft dam as to that of a 700-ft dam.

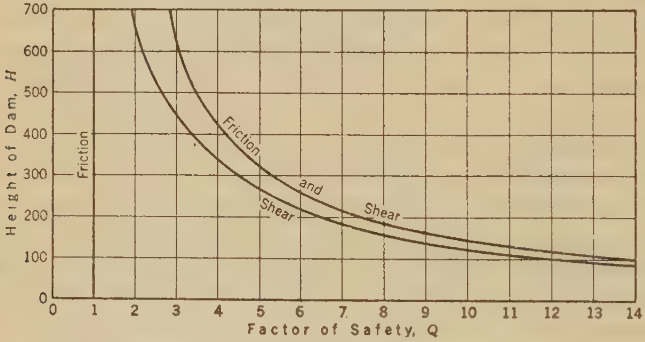

Fig. 9.—Effect of Height of Dam on Factor of Safety Against Sliding

The factors of safety, Q, in Table 8 are tentative pending "the results of additional research." In the writer's opinion they are too great unless certain precautions are taken. Mr. Henny finds that experiments indicate a value of 800 lb per sq in. for shear. This he has reduced tentatively to 400 lb per sq in. on account of uneven shear distribution which he uses in his example. The value of 400 lb per sq in. may be reasonable for shear in solid concrete, but it is not applicable to joints.

It is obvious that shrinkage of new concrete when placed on old concrete or rock foundations will impair, greatly, the adhesion and shearing strength.

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[20] *Engineering News-Record* December 4, 1930, p. 884.

[21] Chf. Engr., The Power Corporation of New York, Buffalo, N. Y.

[22] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 197.

As an example, Joseph Wright, M. Am. Soc. C. E.,[23] found that only 4% of the area of the base of a large pier had bonded with the rock surface on which it was carefully placed.

Thus, it is obvious that shear cannot be relied upon unless all joints, including those at the rock surface, are properly keyed and the sectional area of keys alone are considered as resisting shear. With the most efficient key system, the effective area of shear at any joint would be 50% of the joint area. Applying the author's 400 lb per sq in. on 50% effective area of joint results in the use of 200 lb per sq in. over the entire area. This adjustment reduces, greatly, the factors of safety, Q, in Table 8, resulting in $Q = 1.92$ for a 700-ft dam, as compared with 4.24 and 7.51 for a 200 and a 100-ft dam, respectively, in the case of heel loading equal to $0.44\ H$.

If one could be sure of the 400 lb per sq in. adopted as the shearing value in solid concrete blocks, these factors of safety would be sufficient even for a 700-ft dam. In this case simply roughening the foundation (if not already provided by Nature), and also the joints in the concrete, should be ample for very low dams. Medium high dams up to, say, 200 ft, should be provided with exceedingly rough or keyed foundations and bonding stones in the concrete joints. Above that height, keys in both the foundation and concrete joints should be provided. Dams of greater height than those already built lead into unexplored territory, and their design should be approached with caution, based on more adequate shear experiments.

It is interesting to note, although not the subject of this paper, that the factor of safety against overturning, to be as great as that provided in all other structures, must rely upon tension, and that the additional factor of safety provided by tension is reduced, for high dams, in much the same manner as that indicated by the author for shear.

As the criteria for the uplift force, Mr. Henny uses: (1) A unit uplift pressure evenly distributed from full head-water pressure at the heel to zero (no tail-water) pressure at the toe; and (2) an uplift force equal to the product of the unit uplift pressure and the area of voids in a given section.

The first criterion is safe enough provided care is taken to make the dam less pervious at the heel than at the toe. As for the second criterion, experiments indicate a void volume of about 10 or 12% when subjected to absorption tests. However, the void area in any section may greatly exceed the void volume. Moreover, the absorption tests would not indicate voids that would be penetrated under high pressure. Therefore, the author's use of 25 to 40% for void area is commendable notwithstanding the inadequacy of his "small dams" tests on which it is based. The small dams might have adhered to the rock sufficiently to prevent "progressive increase of percolation or uplift pressure" even if the voids exceeded the percentage assumed.

Defects in the foundations, such as horizontal seams, are the great sources of uncertainty. In the Austin, Pa., dam, the friction of a clay-filled horizontal seam in the foundation was not adequate, but resistance was provided by the horizontal-column action of the layer of rock between the dam and the seam which extended a considerable distance down stream. When this column was cut through by the scouring overflow, the dam slid on the seam. The required resistance ("toe hold") provided by the rock down stream from the dam, varies as the total water pressure or as the square of the height of the dam. In other words, the thickness of the layer between the dam and the seam, if determined by the horizontal crushing strength of rock, must be forty-nine times as great for a 700-ft dam as for a 100-ft dam; and if it is determined by the weight of the layer, it must be forty-nine times as heavy for a 700-ft dam as for a 100-ft dam.

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[23] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 207.

F. W. Hanna,[24] M. Am. Soc. C. E. (by letter).—Many engineers familiar with, and experienced in, the design of dams will welcome this paper on the stability of straight concrete gravity dams even if they may not feel fully inclined to adopt the shearing strength as the principal basis for the factor of safety of such structures. The writer has found that many designers of dams consider the shearing strength on the foundation and other horizontal sections of a concrete dam as an added factor of safety against sliding, but that most of them feel constrained to use the tested wet friction coefficient as giving the minimum resistance to sliding on horizontal planes, particularly at the foundation. The ultimate shearing value used, however, is the unloaded shearing strength of the foundation or dam material, taken as 50%, or less, of the 28-day compressive strength with an appropriate factor of safety and applied uniformly from heel to toe over the horizontal section under consideration. The use of 50% is warranted by experiments made by Professor Arthur N. Talbot.[25]

In the ordinary gravity dam the vertical loading due to the weight of the dam and to the water pressure, and, consequently, the shearing strength, may be assumed to vary uniformly from a small amount at the heel to a maximum at the toe when the reservoir is full and *vice versa* when it is empty, with intermediate conditions of loading for intermediate stages of water levels. For this reason the minimum shearing strength should be found and used.

Equation (3) may be written (using the author's nomenclature and pressure assumptions), as follows:

$$S = S_1 + k\left(\frac{aHL}{2} - \frac{chL}{2}\right) \dots\dots\dots\dots\dots\dots (17)$$

in which, h is the depth of water acting on the up-stream face of the dam, and c is the combined coefficient of uplift area and pressure intensity at the heel. Inspection of Equation (17) shows that the total shearing strength on a horizontal section increases as the value of h decreases and that, therefore, the total shearing resistance along any horizontal section is a minimum for full reservoir for a specific coefficient of uplift. It also shows that the total shearing strength is a minimum for a value of unity for c; that is, for a maximum uplift force. The total shearing resistance therefore, should be based on full reservoir pressure and the maximum probable value of c. For full reservoir, $h = H$, and Equation (17) may be written,

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[24] Ankeny, Iowa.

[25] "Tests of Concrete: I, Shear; II, Bond," *Bulletin No. 8*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

$$S = S_1 + \frac{k\,H\,L}{2}\,(a - c) \qquad (18)$$

For $c = 0.5 \times 0.4 = 0.2$ and $a = 2.4$, as suggested by the author, it will be seen that the last term of Equation (18) is decreased only about 8% by the consideration of uplift in the shearing strength and, since this term is a relatively small part of the entire right member of the equation for ordinary working pressures, the effect of uplift on the total shearing strength is relatively small. This is specifically true because of the existing uncertainty as to the unloaded shearing value of concrete as determined by tests. Tests by Professor Talbot[25] show: A mean of 820 lb per sq in., with a variation from maximum to minimum of 289 lb for plain plates; a mean for recessed blocks of 932 lb, with a variation of 343 lb; a mean for reinforced recessed blocks of 1 476 lb, with a variation of 504 lb; and a mean for restrained beams of 1 167 lb, with a variation of 293 lb.

The minimum unit shearing strength in any horizontal section for reservoir full is at the heel where the net unit pressure on the foundation is nearly zero. If the weight of concrete and the uplift force and its distribution are assumed to be fully known, the theoretically perfect design of the author's triangular dam would make the foundation pressure at the heel exactly zero. In any good design it would not be much in excess of this value. In such a case the minimum unit shearing strength of the horizontal section would be at the heel for reservoir full and would be represented by the unloaded unit shearing strength of the concrete. If shearing rupture occurred at this weakest point, progressive increases would occur across the entire horizontal section and failure would result. For this reason the writer uses 250 lb per sq in.[26] for unit horizontal working shearing strength, this amount being about 25% of the ultimate unloaded shearing strength of 1:3:6 concrete.[25]

The author has accumulated data on uplift pressure that are quite helpful. His assumption of 0.5 H is commendable for reasonably good rock foundations and concrete. For gravel foundations often encountered in the construction of diversion dams the uplift pressure may approach unity, particularly where lengths of down-stream aprons rather than extensions of up-stream cut-off walls to impervious materials are relied on to reduce uplift pressure.

In the determination of uplift areas the writer prefers to use the following formula:

$$c' = 1.21\, v^{\frac{2}{3}} \qquad (19)$$

in which, c' is the area of a great circle of an assumed spherical void, and v is the volume of such a void. Expressing the relation between the volume of the assumed spherical void in terms of its radius; expressing the area of a great circle of the spherical void also in terms of its radius; and, eliminating the radius between these two equations, Equation (19) results. Although it is probable that the voids in concrete or rock have a multiplicity of shapes, they may be assumed to be spherical as a general average. It is probable also that a section of maximum voids through the concrete will be a warped surface passing through the largest number of pore spaces. However, for computation purposes the holes cut by this warped surface may be considered to be projected on to a horizontal plane. Now, it is evident from Equation (19) that, if v is taken as the proportion of the volume of the concrete occupied by voids, c' will then represent a corresponding proportion of the area of the horizontal section occupied by the area of the projected voids on it. Applying Equation (19) on the basis of this assumption to the author's uplift areas given in Line 2 of Table 8, the following values of v and of the estimated relative proportion of voids in the concrete are found:

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[26] "The Design of Dams," by Hanna and Kennedy, p. 119.

Void area, c'	Volume of void, v
0.40	0.19
0.375	0.17
0.35	0.16
0.325	0.14
0.30	0.12
0.275	0.11
0.25	0.09

The values of v obtained correspond reasonably well with the author's assumed 10 to 12% of voids.

In Table 6, Mr. Henny lists some assumed effective uplift pressure areas for dams founded on good rock for vertical loading at the heel, and he gives "somewhat arbitrary" effective uplift pressure areas for increased vertical loadings. For heel loadings varying from $0.4\,H$ to H this listing indicates that the effective uplift area decreases from 40 to 25 per cent. It appeals to the writer that this is too great a decrease, and that the decrease according to Hooke's law should be that due to increased deformation of the concrete caused by increased vertical load on the walls of the individual voids. The horizontal load would consist of two parts: (1) A constant load, H, arising from water pressure on the water face of the dam; and (2) an increased load arising from the increased vertical load acting through Poisson's ratio. The increased vertical heel loading under consideration is $H - 0.4\,H = 0.6\,H$. For concrete with a modulus of elasticity of 3 000 000, the vertical shortening of the voids with diameters, d, in a unit area of horizontal section would be, $0.6\,H\,d \div 3\,000\,000$, and the lateral shortening would be about one-fourth of this amount, or $0.6\,H\,d \div 12\,000\,000 = d(H \div 20\,000\,000)$. For a head of 200 ft the change in d would be $d \div 160 = 0.00625\,d$, or a little more than 0.5% of d. This should be somewhat representative of the contraction of the pore area due to change of loading. On the other hand, the author's reduction from 40 to 25% for the change of pressure area reduces it by $37\frac{1}{2}$ per cent.

The author's application of the shear safety factor to curved gravity, gravity arch, and arch dams is an interesting procedure and provides an excellent check on the general stability of dams against down-stream movement. As far as the writer knows, the idea of considering shearing resistance to rotary movement about a vertical axis in or near one end of the dam is novel. The writer has used the unloaded shearing resistance of the contact area of the entire dam as a balancing force against water pressure in all types of dams.

That the shearing resistance is a good criterion for determining the safety of well constructed dams on good sound rock foundations, with natural or artificial roughening, is feasible, is unquestionable; but its use for poor construction and poor foundations is not advisable. It is also thought that the use of the uplift force in determining the shear factor of safety is an unwarranted degree of refinement, and that the decreases in uplift area for increasing reservoir pressures are too large.

Lars R. Jorgensen,[27] M. Am. Soc. C. E. (by letter).—Undoubtedly, the shearing strength of the concrete in a straight gravity dam may add considerably to the factor of safety of such a structure, especially if this shear is accompanied by compression in a plane perpendicular to the plane of shear. As a general rule, dams are now constructed to take advantage of all the possible shearing strength of concrete and rock by stepping the surfaces to resist a down-stream movement in the best possible manner.

When a material is subject to the ordinary compression test, there is pure compression in the direction of the axis of the testing machine and zero stress in directions normal to it. In all other directions there is both compression and shear. The shear strength and the compressive strength are interdependent. When a specimen is crushed in a compression test it is said to have failed in compression, but that does not imply that the actual failure was compression, because it may just as well have been brought about by shear, and for concrete this is generally the case.

There is fair agreement among engineers that concrete is not reliable in tension. Both the Bouzey and the Habra Dams failed above the foundation, and at the section where failure occurred a small tension was calculated. This was computed on the usual assumption of linear stress distribution and on the further assumption that there was no uplift.

The assumption of linear stress distribution on horizontal planes may be correct for the upper part of the dam, but the writer is convinced that it is incorrect near the base. This was shown quite conclusively by the English tests,[28] and again by some simple tests carried out[29] by Donald P. Barnes, Jun. Am. Soc. C. E., so that all the experimental evidence available indicates non-linear stress distribution at the base. Mohr's assumption of parabolic shear distribution on horizontal planes[30] and Jakobsen's stress distribution according to the principle of least work,[31] both indicate the presence of considerable tension at the up-stream face of a section with about a 66⅔% base-to-height ratio, while the usual theory shows no tension and, furthermore, this is on the assumption that the uplift is zero.

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[27] Cons. Hydro-Elec. Engr., Berkeley, Calif.

[28] "Experimental Investigations of the Stresses in Masonry Dams Subjected to Water Pressure," by Sir John W. Ottley and A. W. Brightmore and by J. S. Wilson and William Gore, *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907–08, Pt. II, Lond., 1908; see, also, for example, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 505, and Figs. 8, 9, 12, 13, 14, 18, and 19.

[29] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 536.

[30] "Technische Mechanik," von Otto Mohr, Second Edition, p. 296.

The Levy section is supposed to have a vertical pressure at every point equal to or greater than the hydrostatic pressure of the water, assuming linear stress distribution on horizontal planes. Therefore, if a seam does exist in the foundation, the water can not enter or can not spread out, because it encounters horizontal surfaces pressed together with a pressure greater than that of the water. Such a section should always be safe, therefore, although it may not be as safe as Levy thought it was on account of actual non-linear stress distribution toward lower elevations. When the base width is less than that prescribed by Levy, the dam is safe only provided the uplift does not exceed a certain definite amount.

The selection of uplift as 40% of the hydrostatic pressure, as advocated by the author, seems rather low; its intensity depends upon so many factors about which the engineer can know little. When the stresses due to earthquake shocks and the fact that nothing very precise is known about the stresses at the base are included, the conservative designer will hesitate to depart very far from the Levy section when designing an important structure.

The effect of uplift is well illustrated in Fig. 5. Fig. 5(*a*) probably represents a favorable case and Fig. 5(*b*) represents a case more severe than it would ordinarily be necessary to assume. The uplift pressure cannot act on the full base area, and the author chooses 40% of this area as the maximum. In order to inject a factor of safety into this uncertain matter, the late James B. Francis, Past-President and Hon. M. Am. Soc. C. E., recommended that the total base area be used for the calculation of uplift. Such a factor of safety seems very sensible.

The Levy assumption, represented by the case in Fig. 5(*b*) (except that $X = 0$) lies between the two extremes illustrated in Fig. 5(*a*) and Fig. 5(*b*); it therefore seems logical not to discard the Levy assumption when designing gravity dams of medium height, except for dams on the very best of foundation materials.

Since the effect of uplift is largely a matter of judgment, a lower and an upper limit of base width for gravity dams could well be decided upon by the profession for the guidance of State engineers or other regulatory bodies. The lower limit of base width would be for dams on excellent and tight foundations, and the upper limit would be for those on seamy rock not so solid. The judgment of the engineer would then decide each intermediate case. With concrete weighing 150 lb per cu ft, it seems reasonable to make these limits of base width 75% and 85% of the height of the dam, respectively, for dams of moderate heights of triangular cross-section.

It is true that a number of gravity dams have a base-to-height ratio of about 66⅔, and when zero uplift is assumed, no earthquake shocks are allowed for, and linear stress distribution is assumed, such sections have a factor of safety of one against overturning and it is quite immaterial what their factor of safety is against sliding, provided it is greater than one. In order to be able to show a factor of safety for such sections materially greater than one, it will be necessary to count on the shearing strength and the tensile strength of the concrete or bond to rock. To expand such a section to a base width of 0.75 H will require about $12\frac{1}{2}\%$ more material and, perhaps, 10% additional cost. This seems a small difference.

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[31] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 489.

The Pine Canyon Dam on the San Gabriel River, in California, a dam of moderate height, has a base width of 86% of the height, and a substantial top width. Being built on good rock foundation (although not excellent), this dam is probably the most conservative gravity dam in the West, where there are too many gravity dams with base widths considerably less than 0.75 H, several of which, like the Pine Canyon Dam, are in earthquake zones.

In a dam actually developing arch action, conditions are most favorable to the utilization of the shearing strength of the concrete, as a wedge action is present and the shear surfaces are definitely under compression. The trough-like surface along the rock contact can then withstand considerable shear with safety, as mentioned by Mr. Henny, since shear action can be depended upon in combination with compression.

Most dam failures, in the past, could have been prevented by designing a heavier section. The behavior of the St. Francis Dam, as referred to by Mr. Henny, shows this clearly. The end portions of this dam, with base widths of 0.6666 H, promptly failed when the water level in the reservoir had nearly reached the top, whereas the middle portion of the dam, having a bottom width equal to about $82\frac{1}{2}\%$ of the height stood. There was no shearing or tensile strength to help hold the cantilever down because there was no bond to the poor foundation. Such bond would have been necessary to keep the ends in place. There was no shear or bond strength available in the middle either, but this was not so necessary there on account of the heavier section. There is no substitution for weight in a gravity dam, except perhaps the addition of buttresses on the down-stream side, but so far this expedient has not been very popular. To the writer it would seem quite logical and inexpensive to increase the overturning resistance on dams of medium height in this way as no gravity dam has failed initially by the crushing of the down-stream toe.

The deflection curve for a triangular cross-section of a cantilever under water load is approximately a circle, and the maximum deflection at the top of the triangle is $\frac{62.5\,H^5}{E\,L^3}$, using the author's notation. For such a section with $H = 200$ ft, say, the deflection at the crest would be four times larger than with $H = 100$ ft. For a dam in an inverted A-shaped canyon, the cantilever will deflect more in the middle at the crest than on the side-hill. The horizontal beam will attempt to neutralize this tendency by transferring the load from the middle high portion of the dam toward the shorter cantilevers on the side-hill. For equal safety the section on the side-hill, therefore, should be heavier than the section across the middle high portion of the opening. The writer mentions this specifically because little attention has been paid to this feature in the past, and it should always be the aim of the designer to produce a structure having the same factor of safety in all its parts. An arched gravity dam is worse in this respect, than a straight gravity dam, since the ends of the arched gravity dam generally have to be placed on a down-hill slope, and designers of gravity arched dams similar to the St. Francis Dam cannot rely on help from the arch at the season of greatest contraction (March).

The author does not advocate slimmer sections than those now in use, but it also seems that he does not point out the absence, to all intents and purposes, of a factor of safety in a gravity dam section with a base width of 66⅔% of the height, in case the tensile and shearing strength of the concrete should fail through cracking or should be diminished with age.

A dam with such a section may be on the verge of failure, and 10% added to the cost of such a structure most likely will mean the difference of its success or failure. The St. Francis Dam was a practical demonstration of the behavior of a dam section with a base width of 0.6666 H in one place and a section with a base width of 82½% of the height in another place. About 10% added to the cost of this dam could have expanded the base width of the ends (that failed) to 82½% of the height at the rock contact, the same as for the middle portion, it would have deepened the excavation, and undoubtedly would have prevented the tremendous loss involved. The engineer must consider that in one case he is practically "flirting with the undertaker" and, in the other case, he is, under ordinary circumstances, producing a dam of adequate safety.

The Engineering Profession should establish an allowable minimum base width for a gravity dam and this should be considerably in excess of 0.6666 H. There would still be plenty of room for the exercise of judgment by the individual engineer in deciding how much more than the allowable minimum the base width should be for his structure.

I. M. NELIDOV,[32] ASSOC. M. AM. SOC. C. E. (by letter).—When a gravity dam is acted upon by the pressure of water, the resistance due to shear should be introduced in addition to the resistance due to sliding friction. This is the substance of Mr. Henny's paper.

The formula given for total resistance per unit of area is: $s = s_1 + k_q$, and a factor of safety for the entire horizontal section of $Q = 4$ is adopted. Uplift loading, including the variation of its intensity with vertical loading, and the area of its application are also discussed.

It must be recognized that for about fifty years there has been little advance in the knowledge of the proper design for the stability of structures in general and of dams in particular. The importance of obtaining reasonable values for stability factors should not be minimized, especially if comparison is made with those of stresses. For instance, with compressive stress in concrete of 500 lb per sq in., the factor of safety is at least 4. At the same time, while computing stability against horizontal forces, with the usual sliding ratio of about 0.6 and disregarding shear resistance, a safety factor of only 1.3 is obtained. A safety factor in overturning equal to 1.5 is considered very satisfactory in case the resistance due to bond with the foundation is not taken into account. With uplift both factors attain still lower values. This indicates to what extent stability factors are smaller than those of stresses. On the other hand, raising stability factors to the same value as stress factors would result in excessive dimensions, provided no resistance other than sliding friction and the resisting effect of dead weight is included.

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[32] Senior Engr. of Hydr. Structure Design, State Dept. of Public Works, Sacramento, Calif.

As shown in the paper the inclusion of shear resistance gives an increase of safety factor from about 1.3 to 4, thus indicating that the greatest part of the resistance is supplied by this newly included element. This condition requires a serious study of these new elements, since the safety of the dam depends entirely on the correctness of the values selected. An investigation of the ultimate resistance due to shear, sliding, and bond should be made for planes of contact of concrete with rock, at construction joints, and within the material itself. The fact that the rock structure is not uniform and that planes of weakness may exist hidden from the investigating eye, must be taken into account. This, together with saturation of the foundation rock, should warn against the haphazard use of formulas including resistance due to shear and bond. Sound design should be based upon carefully selected test data and reasonable safety factors approaching 4 should be attained. It is wasteful to neglect the resistance if it can be relied upon. During the construction of Hogan Dam in 1929, the writer used data on shear resistance of horizontal joints obtained by Mr. L. H. Tuthill,[33] but was very cautious in applying them due to the singularity of the test. The fact that stability can be altered so greatly by the inclusion or the omission of the shear element seems to justify this attitude.

The process by which the values, s_1 and k, are obtained is necessarily approximate. First, the assumption of a plane of rupture is far from what actually occurs in a cylindrical body; and, second, the coefficient of friction is not constant with normal stress. This makes Equation (4) an unknown of a high degree. Professor S. Timoshenko[34] states that if the surfaces of contact of concrete specimens are covered with paraffin, the ultimate strength will be greatly reduced and the type of failure will be completely different. A solid cube fails by dividing into planes parallel with one of the lateral sides. This indicates that the mode of loading will change the character of failure entirely. Besides, the actual condition of loading is far from that produced in a laterally unsupported cylinder as used in concrete tests. Consequently, the values of resistances, s_1 and k, are not representative and may not even be correct. It appears that tests similar to those by Mr. Tuthill, in which a middle block was displaced relative to the side blocks, should supply more correct values for shear and sliding resistance. The segregation of shear and sliding proper can be made by testing the displaced specimen for frictional resistance only.

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[33] "Horizontal Joint Shear in Mass Concrete," by Lewis H. Tuthill, *Western Construction News*, November 25, 1931.

[34] "Strength of Materials," by S. Timoshenko, Pt. II, p. 655.

Experiments quoted by the author and those by Mr. Tuthill indicate that total horizontal resistance increases with the normal pressure to a certain value, beyond which it begins to fall. Crushing of particles may be responsible for this fact. This may also raise a question concerning the simultaneous action of shear and sliding. A negative answer to this question would disprove the theory based on the combined action. Additional experimental data will supply the answer.

In passing, the writer wishes to remark that it would appear logical to use formulas for stresses which take into account tension in concrete when the total width of the base is taken in computing sliding-shear resistance.

To complete a study of stability, overturning also must be discussed. Assuming no bond between the base of the dam and foundation, the stability is assured only by the resisting moment of the weight and other vertical forces acting downward. In a general case there may be four sums of forces affecting the stability in overturning, namely, the vertical forces acting downward and upward and two horizontal forces acting to the right and to the left. The first sum, W, results from the weight of masonry, vertical water pressure, and weight of débris on the up-stream and down-stream faces; the second, U, is produced by uplift; the third, P, by the horizontal water pressure and débris on the up-stream face; and the fourth, P_1, by the horizontal water pressure on the down-stream face. The factor of safety in overturning, as in the case of the factors of safety in sliding, tension, shear, etc., must express the ratio of the ultimate overturning ratio (at which overturning is bound to occur) to the overturning ratio produced by the acting forces in the case considered. Overturning ratio is defined here as:

$$r_0 = \frac{\Sigma M_r}{\Sigma M_0} > 1 \dots\dots\dots\dots (20)$$

in which, ΣM_r is the moment that resists overturning; and, ΣM_0 is the moment that produces overturning. At the point of overturning the ultimate overturning ratio is:

$$r_u = \frac{M_r}{M_u} = 1 \dots\dots\dots\dots (21)$$

in which, M_u is the moment at which actual overturning occurs.

The factor of safety against overturning will be:

$$F = \frac{r_0}{r_u} > 1 \dots\dots\dots\dots (22)$$

According to the definition of N. Kelen[35]: $M_r = M_w + M_{p1}$; and, $M_0 = M_p + M_u$; or,

$$F = -\frac{M_w + M_{p1}}{M_p + M_u} \dots\dots\dots\dots (23)$$

in which, positive moments are clockwise.

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[35] "Die Staumauern," von N. Kelen.

If a dam is anchored to its foundation, or if bond between the concrete and rock can be relied upon, these new elements will increase the potential resisting moment within the limits of their own factors of safety. Consequently, consideration should be given to the existing and allowable factors of safety of the anchorage. When the actual factor of safety of the anchorage is smaller than the allowable, the anchorage will be unreliable until the stress in it will be lowered to such a value that the factor of safety will become equal to the allowable factor. For example, using the author's notation, select a dam with $H = 150$ ft; $L = 116.6$ ft; $n = 0.777$; and $a = 2.4$, which corresponds to the curve in Fig. 8 with $Q = 4$.

For the condition of no uplift: $l_1 = 48$ lb per sq in.; $l_2 = 108$ lb per sq in.; and $F_1 = \frac{M_w}{M_p} = 2.90$. If the ultimate bond stress is taken at 150 lb per sq in., then, with a factor of safety of 4, only 37.5 in-lb may be allowed for safe tension. The additional overturning moment which will bring the stress to this value will be:

$$M'_o = \frac{1}{6}\,(37.5 + 48)\,144 \times 150^2 \times \frac{1 + 0.777^2}{2 + 0.777^2}\,2 \times 0.777^2 = 34\,524\,000 \text{ ft-lb}$$

The new factor of safety will then be:

$$F_2 = \frac{M_w + M'_o}{M_p} = 3.88$$

For the condition of 50% uplift at the up-stream face (32.5 in-lb) and 0 at the down-stream face: $l_1 = 48 - 32.5 = 15.5$ lb per sq in., and $l_2 = 108$ lb per sq in., which assumes that bond between the concrete and the rock was not broken by the uplift force. Disregarding the reserve due to the bond, the factor of overturning will be:

$$F_1 = \frac{M_w}{M_p + M_u} = 1.81$$

The additional overturning moment that will bring the stress to the maximum allowed (37.5 in-lb) will be:

$$M'_o = \frac{1}{6}\,(37.5 + 15.5) \times 144 \times 150^2 \times 0.746 = 21\,432\,000 \text{ ft-lb}$$

This will make the factor of safety, including bond, equal to:

$$F_2 = \frac{M_w + M'_o}{M_p + M_u} = 2.19$$

If Mr. Henny's assumption is followed in that the bond between the concrete and the rock is broken by uplift, the stresses will be determined by Equations (10) and (11) thus: $x = 74.5$ ft; $l_2 = 137$ lb per sq in.; and $l_1 = 0$. In this case:

$$F_1 = \frac{M_w}{M_p + M_u} = 1.10$$

The foregoing computations indicate that factor in overturning may be one of the weak spots in stability.

PAUL BAUMANN,[36] M. AM. SOC. C. E. (by letter).—The crushing test, simple though it seems on the face of it, is in fact very intricate. It is far from being satisfactorily interpreted by means of Coulomb's internal friction or wedge theory, which dates back to the third quarter of the Eighteenth Century and which is closely related to the wedge theory of earth pressure and earth resistance, respectively.

A somewhat later interpretation of the crushing test has been based on the postulate that the failure occurs along surfaces of maximum shear (principal shear), which leads to the contention that the shearing strength is one-half the first principal or direct stress in a mono-axial system. Both theories are approximations, the latter being fairly close for cubic specimens, but both gain in fallacy with a decrease in the restraint of the faces in contact with the metal shoes of the testing machine.

Due to the friction between concrete and steel and due to the much greater rigidity of the latter, the concrete is restrained from transverse expansion (Poisson's ratio) which causes a curvature of the sides and a stress condition which is no longer mono-axial.

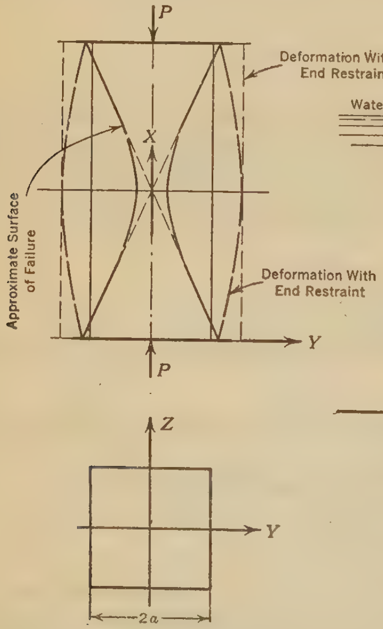
P
Deformation Without End Restraint
X
Approximate Surface of Failure
Deformation With End Restraint
Y
P
Z
Y
2a

FIG. 10.

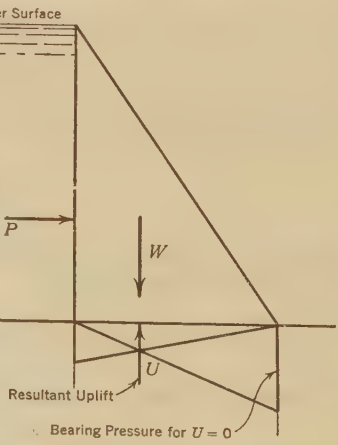
Water Surface
P
W
U
Resultant Uplift
Bearing Pressure for $U = 0$

FIG. 11.

Prior to failure, the specimen thus takes a shape as shown exaggerated in Fig. 10, and failure starts half way between the shoes where the effect of end restraint and the density of the material are a minimum. The shape of the surface of failure is the reflected shape, so to speak, of the free surface of the specimen. Tests conducted by Foeppl[87] showed that, with the contact faces of the specimen well lubricated, failure occurs along planes parallel to the main (vertical) axis and divides the specimen into vertical prisms. The crushing strength in some cases was reduced to one-fourth that for non-lubricated contact faces. It would be rather difficult to explain this phenomenon by means of Coulomb's wedge theory.

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[36] Chf. Designer, Quinton, Code & Hill-Leeds & Barnard, Engrs, Consolidated, Los Angeles, Calif.

Similar results were obtained a short time ago with concrete cylinders of various sizes by J. Y. Jewett, Assoc. M. Am. Soc. C. E., in the testing laboratory of the City of San Diego, Calif.

A tentative study by Foeppl[88] on the crushing test without lubrication leads to the conclusion that the contact faces are only partly restrained and that the distribution of normal and tangential stresses there (Fig. 10) may be expressed as:

$$\sigma^{o}_{x} = -\frac{P}{4\,a^2} + 0.72\,C\left\{\cos\pi\frac{y}{a} + \cos\pi\frac{z}{a}\right\} \quad \ldots\ldots\ldots\ldots(24)$$

and,

$$\tau_0 = C\sqrt{\sin^2\pi\frac{y}{a} + \sin^2\pi\frac{z}{a}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(25)$$

in which, C is a constant that depends on the coefficient of friction between steel and concrete and which follows from the conditions, $\frac{\partial\left(\frac{\tau_0}{\sigma^{o}_{x}}\right)}{\partial y} = 0$ and $\frac{\partial\frac{\tau_0}{\sigma^{o}_{x}}}{\partial z} = 0$; and that the maximum value of $\frac{\tau_0}{\sigma^{o}_{x}} = \mu =$ coefficient of friction between the steel and the concrete.

The author very ably treats the uplift problem from a standpoint of inner stability rather than overturning. As long as the unit weight of the dam is greater than that of the liquid it retains, and as long as the uplift pressure is governed by the water surface in the reservoir and the dam cannot slide without shearing, the influence of uplift is on the inner stability only, because no change due to uplift of the forces above the base of the section under consideration has taken place. If the resultant uplift force were replaced by the pressure of a jack (Fig. 11), the outer stability would certainly not be changed as far as overturning is concerned, but the distribution of normal and tangential stresses along the section would have changed due to a rotation of the section and a diminution of contact, respectively.

Levy's rule to make the compression in the concrete at the up-stream face at all elevations at least equal to the water pressure under maximum head has been taught in leading engineering schools of Europe as the rule and has been the basic design principle of nearly all the straight or quasi-straight gravity dams built since it was first advanced. Although this may be too severe a practice, it certainly helped to minimize failures and it should always be observed in locations subject to seismic disturbances.

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[87] "Verlesungen über Technische Mechanik," Vol. 3, Tenth Edition, pp. 87-88.

[88] "Drang und Zwang," Vol. 1, Second Edition, p. 116.

The stress function of a gravity dam of triangular shape and infinite height and length is known, and through it normal and tangential stresses may be determined at any point; these are the criteria of structural safety. For a finite height (which, of course, is the case in practice), the influence of yielding foundations must be superimposed. This influence also affects the uplift studies. It is particularly noticeable at the heel and toe and relatively small changes there may greatly reduce the stresses which, for a right angle between up-stream face and rock surface, tend to assume alarming magnitudes. These studies are best made by means of photo-elastic tests on models.

With intelligent construction, the sliding factor is of less importance than the shear safety factor, except at the foundation in the case of a slope in a down-stream direction. The occurrence of resistance due to shear is then doubtful in most cases, while the existence of resistance due to a coefficient of friction of, say, 0.70 cannot be doubted, barring abnormal uplift conditions.

THADDEUS MERRIMAN,[39] M. AM. SOC. C. E. (by letter).—The presentation of uplift, based on the experiments of Mr. Woodard, is an especially interesting part of this paper. Undoubtedly, pore-space pressures are present in all dams and act in an upward direction with magnitudes of the order stated. That being the case, these pressures must also act horizontally and the load, *P*, in Fig. 5 may not be considered as applied on the water face of the dam. In other words, to whatever extent the pores of a dam are filled with water under pressure to that extent will the total resultant of the water loading be applied in a diagonally upward direction within the body of the structure.

It is not easy to follow the author's logic with respect to the subject of shear because it is more than probable that every failure of a brittle material is actually a failure in tension. As yet the science of materials has not been advanced sufficiently to enable one to think with certainty in terms of shear which, to this time, has eluded definition. The author, however, assumes the correctness of Coulomb's internal friction theory and combines it with a proposed "basic formula" which is simply a ratio of convenience between two total values, the one a shearing strength, the other a water loading. From the vantage point of a new formula thus constructed, he then shows in Table 9 that the "shear safety factor" for the Elephant Butte Dam is 7.3, whereas the "apparent factor of safety" is 1.27. Which of these values, if either, is correct and which of them best represents the actual stability of this structure?

Much of the present-day confusion in regard to the analysis of dams is attributable to the use of a concept which seeks to express a purely physical situation in terms of an abstract view of relative safety. The true stability of a dam is not to be stated as simply as by any "factor of safety."

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[39] Cons. Engr., New York, N. Y.

IVAN E. HOUK,[40] M. AM. SOC. C. E. (by letter).—An excellent treatment of straight, concrete, gravity dam design is contained in this paper. The methods proposed for considering foundation uplift, pore pressure, shear resistance, and the total factor of safety of such structures against failure by sliding undoubtedly will find favor among those engaged on dam design and construction.

Engineers have long recognized that the principal factor of safety of a straight concrete dam against failure by sliding at the base lies in the shear resistance of the concrete and foundation materials rather than in the amount by which the coefficient of friction exceeds the calculated sliding factor. Shear resistance has always been important at the base of a dam, because of the fact that the concrete is poured on irregularly excavated foundation surfaces. During recent years shear resistance has also become important at horizontal construction joints above the base, due to the increased care in preparing joint surfaces during construction, the provision of adequate horizontal keyways at joint elevations, and the use of upward sloping joint surfaces near the down-stream face of the dam.

Undoubtedly, the science of dam design and construction has reached a stage where shear resistance should be considered more carefully in the case of important gravity structures. Because of relatively large available strength in shear it may be possible in certain cases to allow a slightly higher sliding factor than would otherwise be permissible. The writer is whole-heartedly in accord with the proposal to calculate shear-friction factors of safety in designing gravity dams. However, he does not believe that sliding factor calculations should be dispensed with entirely, even in important and costly structures where the designing engineer can feel reasonably sure of securing satisfactory construction operations, rigid inspection, and strict adherence to specified design details. Sliding factors should be computed in the future as they have been in the past; and the cross-sections of all straight gravity dams should be proportioned so that the calculated sliding factors will not exceed the coefficient of friction, even if relatively high shear resistances may be available in all parts of the structure.

There is something wonderfully reassuring about a low sliding factor. Only three factors are involved in its calculation: Water pressure, weight, and uplift. Of these, uplift is the only one that is not susceptible of exact determination. Consequently, if the foundation rock is satisfactory, the designer, by including a conservative allowance for uplift in his calculations, and proportioning the cross-section so that the sliding factor does not exceed the coefficient of friction, can be confident that the dam will not fail by sliding, regardless of any accident, oversight, or negligence in construction operations.

This may seem ultra-conservative. Nevertheless, the writer believes in being ultra-conservative in designing structures the failure of which may mean the loss of many human lives as well as much valuable property. The failure of the St. Francis Dam, in Southern California, a few years ago, although primarily due to poor foundation materials rather than to a lack of shear resistance or a high sliding factor, certainly illustrated the advisability of being ultra-conservative in high dam design. Moreover, the writer knows of a few comparatively high dams, built during recent years, in which, the horizontal construction joints are not capable of developing a high shear resistance. They are safe because they were designed with relatively low sliding factors.

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[40] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

A. V. KARPOV,[41] M. AM. SOC. C. E. (by letter).—An interesting and timely symposium on modern data and ideas as to the shearing strength of concrete and the influence of uplift force on stability of dams is presented in this paper. At the same time the paper is a valuable expression of the dissatisfaction that has been felt for a long time with the present methods of dam design.

The unscientific and highly unsatisfactory assumptions on which the present design of dams is based are getting more and more obvious. The assumption of linear stress distribution and the middle-third theory that presumed the absence of any tension stress in the dam was the beginning. Next came the Levy requirement that was, and still is, popular with European designers. Under this requirement, assuming again the linear distribution of stress, not only was the tension stress to be avoided, but at no point of the up-stream face of the dam was the compression stress supposed to be lower than the hydraulic water pressure at the corresponding elevation.

In the United States the sliding factor was favored for a long time as a controlling design assumption. The author introduces the shearing strength as a new and more desirable controlling design assumption and proposes the revival of the old Coulomb theory.

It would seem advisable to look at the entire matter in a more general way. First, it is to be realized that a gravity dam is a problem in the three-dimensional state of stress. The present limited engineering knowledge makes a proper handling of such problems very difficult. Consequently. the cardinal and necessary simplifying assumption is made in considering the vertical element of a dam as a two-dimensional stress problem. Other arbitrary assumptions must be added very carefully so as to keep the design reasonably close to actual conditions.

Fig. 12 illustrates the divergence between the actual stress, and the stress evaluated under the different design assumptions; it is only a schematic representation and not to scale, either in the linear dimensions or in stress and shear values. Present knowledge does not permit the exact determination of the actual stress and shear, but is sufficient to make possible a schematic representation of the kind shown.[42]

An arbitrarily chosen vertical cross-section of a gravity dam is shown in solid lines in Fig. 12(*a*); the stress and shear at the foundation of this cross-section are to be considered, under full water load, assuming a two-dimensional state of stress. Fig. 12 (*a*) also shows the vertical stress distribution superimposed. In drawing this diagram it is assumed that the dimensions of the dam are such as to make it fulfill the requirements of the middle-third theory. The dotted line shows the vertical stress at the foundation evaluated under the assumption of linear distribution of stress and the resultant passing through the third of the base; consequently, the vertical stress equals zero at the heel of the dam. The solid line shows the actual vertical stress at the foundation of such a dam.

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[41] Designing Engr., Hydr. Dept., Aluminum Co. of America, Pittsburgh, Pa.

[42] See, also, "Model of Calderwood Arch Dam" by A. V. Karpov and R. L. Templin, Members, Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., December, 1933, p. 1568.

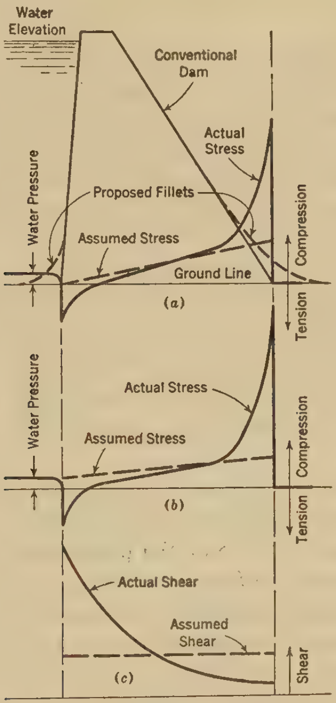
Water Elevation
Conventional Dam
Actual Stress
Water Pressure
Proposed Fillets
Assumed Stress
Ground Line
Compression
Tension
(a)
Water Pressure
Actual Stress
Assumed Stress
Compression
Tension
(b)
Actual Shear
Assumed Shear
Shear
(c)

FIG. 12.—VERTICAL STRESS AND SHEAR AT THE GROUND LINE

Fig. 12(*b*) shows the vertical stress distribution assuming such dimensions of the dam as to make it fulfill the Levy requirement. Again the dotted line shows the vertical stress evaluated under the linear stress assumption (the vertical stress equals the water pressure at the heel of the dam) and the solid line the actual stress. Fig. 12(*c*) shows the shear at the foundation. The author bases his theory on the full-base shearing strength, which is equivalent to the assumption of a uniform shear distribution. The dotted line represents the assumed shear distribution corresponding to Mr. Henny's proposal. The solid line represents the actual maximum shear.

From the study of these diagrams only one conclusion can be drawn; namely, that all three methods are unsatisfactory. As a matter of fact, there is not much difference between them; each one is arbitrary, none is proven, and an attempt to prove them will give results that are contrary to expectations. The author makes an attempt to disregard these facts and to introduce the non-existent imaginary constant shear. An attempt to make such an imaginary shear value a controlling factor in the design does not seem to the writer to be an improvement in present methods. The introduction of a safety factor based on such arbitrary shear value is rather meaningless since the maximum shear may be many times greater than the arbitrarily assumed average shear.

There seems to be a pronounced tendency to forget the fact that in dealing with a large structure, such as a dam, the main question is whether or not the simplified assumptions that are applied to small test specimens may be transferred directly to the design of a dam. A concrete dam is an elastic structure which, depending on the foundation, rests on an elastic or a plastic base. To solve problems in the design of dams properly, recognition must be accorded that simple idea.

The modern tendency in engineering is to utilize the new researches in elastic and plastic problems and to apply them in accordance with modern developments in the theory of stress. As far as the design of dams is concerned, a new theory is not only useless, but may become dangerous, if it does not recognize the fact that, on account of the sharp changes in cross-section and the varying foundation conditions, considerable increase in local stress is possible.

The method proposed by Mr. Henny, in common with other methods, fails to develop the possibility of building up considerable stress concentration if there is an abrupt change in shape at the places where the dam proper meets the foundation. This increase of stress depends not only on the shape of the dam, but also on the elastic properties of the concrete and the foundation, and this stress may become very high on hard rock. The use of proper fillets to reduce this stress concentration is of considerable importance; on Fig. 12(*a*) such fillets are indicated by dotted lines. Any method that claims to be an improvement over the existing methods must reveal that fact clearly.

The importance of this consideration probably is well illustrated by the failure of the St. Francis Dam. This failure is attributed to the properties of the particular foundation rock to disintegrate when saturated with water and subjected to high stress. Such conditions were present and the dam failed. At the toe of the dam the compression stress could have been reduced from two to three times if proper fillets had been introduced. The removal of one of these unfavorable factors—the high stress—probably would have been sufficient to prevent failure or, at least, to make the failure less sudden and thus prevent loss of life and also to reduce property damage.

In order to have all these factors properly introduced in the design, it is necessary to eliminate the arbitrary assumptions and develop a theory that will cover the actual conditions with reasonable accuracy. From this point of view, the paper is of considerable value, since it stresses the present unsatisfactory design assumptions. The writer does not consider that the conclusions advanced in this paper are a step in the right direction. A proper scientific theory, even if rather complicated, is yet to be developed. A real improvement can be demonstrated only if such a theory will pass properly conducted model and prototype tests.

L. F. HARZA,[43] M. AM. SOC. C. E. (by letter).—By presenting an improved and more logical method of design against tension at the heel, the author has made a commendable contribution.

The subject of pore uplift, however, needs further treatment. The Woodard experiments indicate that internal pore pressure exists and that concrete can be fractured in tension by water under pressure entering at one point and escaping in opposite directions through the pores. The seepage water produces a differential pressure on each particle in the direction of seepage, because of the progressive loss in head that causes a greater pressure on the approaching than on the receding face of each particle, and, therefore, a tension if seepage is in opposite directions, as in the Woodard specimens.

An analogous condition does not exist in the pores of a dam, however. In this case, the water seeps in the same general direction, not upward or downward. Assuming approximately parallel and horizontal lines of seepage the condition becomes more nearly analogous to buoyancy; for example, if all the pores on the down-stream face could be closed by a water-tight surface, then static pressure would exist in each pore equal to its depth, H, below the water surface. Bouyancy would be exerted within the concrete to an extent perhaps depending on some unknown function of the porosity. If the dam were composed of uncemented aggregate, full bouyancy would be effective, and the submerged weight of the material would be reduced by the full amount of its displacement, perhaps 80% or more of the gross volume; and yet, the pore space might be only 20%, or much less, of the volume. In this case the surface on which uplift is effective is not a plane cut through the material, but an irregular surface following entirely between the particles.

Likewise with cemented aggregates it may be argued reasonably that the internal bouyancy exceeds the proportion indicated by the porosity. A tension fracture of the material would expose a rough granular surface representing the weakest surface between the particles and not a plane cut through the pieces of aggregate. Now, if the water-tight down-stream face were to be removed, one could reasonably assume an average bouyancy of at least one-half the former, varying from the former value at the up-stream face to zero at the down-stream face, but always greater than that indicated by the porosity.

If the foregoing reasoning is correct, a similar effective proportionate pressure area much greater than the porosity, might assist in explaining the apparent inconsistencies of the Woodard experiments in the author's Table 7.

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[43] Cons. Engr.; Pres. Harza Eng. Co., Chicago, Ill.

The principal inconsistency is more apparent than real. It results because the experiments should be divided in two groups. In the upper group in Table 7, the porosity increases upward as the specimens approach neat cement because impervious sand is being replaced with pervious cement matrix, and yet the strength increases. In the lower group, porosity is increasing downward because of reduction in the cement and is concurrent with reduction in strength. Even with this grouping, quantitative deductions of the laws of change are not possible, perhaps because of the few points in each group and the possible unequal distribution of stress over the cross-section. Even after discounting the results by possible unequal distribution of stress, they indicate clearly, however, that pore pressure is exerted over a much larger proportion of the cross-section than that represented by porosity, especially in the leaner mixes.

The usual assumption that uplift pressure decreases uniformly from heel to toe in a triangular manner presupposes uniform opening of the joints from heel to toe. The greater pressure toward the toe, however, would tend more nearly to close up the joints and increase their resistance, or to reduce the area subject to uplift as assumed by the author. In either case, this would tend toward a trapezoidal or perhaps parabolic rather than triangular variation of water pressure from heel to toe. This might be offset in net effect by silting of the up-stream face.

The author assumes, in Table 6, a buoyancy at the heel of three times, and, at the toe, of twice, that represented by porosity. This is perhaps enough for good concrete in the mass between construction joints, but it is not clear whether the author also intends it to apply to construction joints assuming good workmanship. From the Woodard experiments it would seem at least doubtful whether it can allow for more than the porosity.

The most serious problems of uplift must result from: (1) Lack of intimate contact along construction joints; (2) along the contact with the foundation; (3) bedding planes in stratified rock below the foundation level; or (4), local uplift communicated to isolated areas under the dam through fissures or joints in the rock. All these conditions are defects either in Man's or Nature's part of the structures and cannot yield to any rational basis of analysis. Apparently, allowance for them must forever depend entirely on judgment gained by experience.

Lack of intimate contact at construction joints can readily occur because of the necessity of placing one layer of material, subject to initial volume changes, on a layer already set. Creeping or even warping of the new layer over the old one to an extent sufficient to break the bond is not difficult to imagine as the result of internal stresses set up by these relatively different tendencies to volume change. Such condition would not be revealed by the Bull Run test specimens.

The writer is not quite confident that increase in vertical loading can be assumed to decrease the effective uplift pressure area as indicated in Table 6 in the case of such defective contact. It seems unlikely that a joint once broken can be restored to such intimate contact by pressure as to exclude water. To the extent that such pressure would tend to reduce the opening of the joint, it would cause a greater initial loss of head of seepage water entering the joint and thus would reduce uplift pressure rather than area.

May the writer have the temerity to suggest a radical departure in dam design and construction to eliminate uncertainty from pore and joint pressure and the progressive changes in composition, in some cases approaching disintegration of concrete, due to seepage? Stainless steel and some other metals practically free from corrosion are now available, the former at a cost of about 40 cents per lb. Such metal of a thickness to weigh about 1 lb per sq ft might be rolled with relatively small corrugations, say, 1 in., in one direction and then with large corrugations, say, 3-in. spacing, in the other direction to make the sheets elastic both ways. Such sheets could be used as lagging or alternately inside the lagging on the up-stream face, welded or soldered at the joints and perhaps with special U-shaped seams at all vertical and horizontal construction joints and with frequent spot-welded strips to embed in the concrete and insure adhesion. The cost of such impervious surface would not exceed that of 1 ft or 2 ft of extra thickness of concrete, a negligible amount for a large dam. Its effectiveness should be complete, except where most needed, at the joint with the foundation rock. This problem might be minimized, but never entirely eliminated, by a heel apron similarly surfaced.

The writer fully agrees with the author's general argument for shear factor instead of sliding factor for dams on non-stratified foundations. Mr. Henny admits that sliding factor must still be considered when a foundation contains horizontal stratification. The fact should here be pointed out that the extensive regions of stratified rock, forming a considerable proportion of the earth's surface, do not permit general abandonment of the sliding factor method of analysis, a fact which should be emphasized to avoid possible future misapplications of the author's very valuable contribution to the art. Even in stratified rock the contact surfaces are frequently undulating so that shear as well as sliding will enter in variable amount.

EDWARD GODFREY,[44] M. AM. SOC. C. E. (by letter).—The vast difference in stability of the Boulder Dam and other large dams designed at earlier dates, particularly some that have failed, is ample demonstration of the fact that the design of dams has departed from old standards. Mr. Henny's paper is timely and important, particularly because it comes at a time when engineers are "rethinking" the design of dams. The emphasis placed upon, and the prominence given to, under-pressure in dams in Mr. Henny's paper is quite significant, in view of the fact that not so far in the past it was practically impossible to find any mention of this in the literature of dams.

Mr. Henny emphasizes the importance of considering shear as the prime factor in designing a concrete dam for strength. He also points out the difference in shear strength in concrete that is in tension normal to the shearing plane and concrete that is in compression. This leads straight to the question of tension in the concrete, which, in turn, brings up interior or under-pressure in the body of the dam, or the forces that lift it from its foundation.

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[44] Structural Engr., Pittsburgh, Pa.

A factor that has been neglected in considering the stability of a dam is flotation. Mr. Henny states:

> "When it became evident that failure of solid gravity dams was generally accompanied by down-stream mass movement and that this was facilitated if not induced principally by penetration of pressure water at the base of the dam, or in the foundation, two new factors were taken into account, namely, uplift and sliding."

The down-stream mass movement was never so conspicuously exemplified as in the case of the St. Francis Dam. Great masses of solid concrete, weighing thousands of tons, were moved down the valley for thousands of feet. This is flotation pure and simple. It seems to be a prevailing notion that flotation occurs only when a body is submerged or partly submerged—in other words, when there is water on all sides of it. In the past, engineers who ignored under-pressure, or argued against its inclusion in designing, did not fail to consider flotation or loss of weight of masonry when tail-water was present. The depth of the tail-water was the only head recognized, when the fact is that the greater the head of tail-water, the safer the dam, up to complete safety with tail-water level with the main body (barring flow).

Water on the vertical sides of a submerged vessel has no influence in causing flotation. Water on the down-stream slope of a dam helps to hold it down, rather than to lift it. A submerged body is subject to flotation solely by reason of the pressure of water (or difference in pressure) below the body—in other words, the vertical component of pressure on all surfaces in contact with the water.

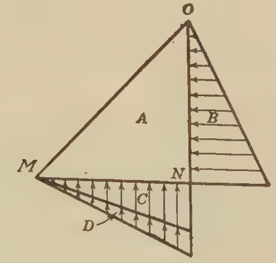

FIG. 13

In a dam in which the water is in contact with the up-stream surface and has access to the base, unrestricted, the flotation is just the same as if it were completely submerged; but the horizontal forces are quite different.

In Fig. 13, *A* represents the cross-section of a concrete dam in which the base is 0.84 of the height; *B* represents the horizontal force of water on the up-stream side; *C*, the upward pressure under the base, full head at *N* tapering to zero at *M*; and *D*, the upward pull against the foundation, forces induced by the water pressure, *B*, tending to rotate the dam about *M*.

In a dam of these proportions, the sum of the forces, *C* and *D*, exactly balances the weight of the concrete, *A*. If it were not for adhesion to the foundation, or if the foundation were not below ground level, this dam would be just on the verge of floating away, because it exerts no pressure on its base, except that which water balances exactly. A dam subject to analogous forces, with a base less than 0.84 of its height (say, 0.65, as in the St. Francis Dam), has every condition favorable for complete flotation. The reaction of the masonry on the foundation is less than the possible pressure of the water. Here is flotation with the addition of a gigantic force to push the dam down stream and a complete explanation of the down-stream mass movement referred to by Mr. Henny.

Under-pressure can no longer be reasoned away. The mere setting up of tests that seem to show its non-existence in some cases has no weight in destroying or controverting the great volume of test and experience that prove its existence in all kinds of cases.

F. KNAPP,[45] ESQ. (by letter).—On the basis of tests cited by the author it can be taken for granted that the internal friction theory as originated by Coulomb is applicable to dam design, at least in the range of sheer stresses usually found. The numerical values for the plane between the dam and the rock foundation remain to be determined. The experiments by S. H. Woodard, M. Am. Soc. C. E., mentioned by Mr. Henny, are not representative of this case. Pure shear tests are practically impossible because such tests always introduce bending moments and hence tension stresses. Since the shear strength under conditions of no load and the "factor of shearing strength increase" probably vary within wide limits for different qualities of concrete and rock, it seems best, under such circumstances, to arrange special tests for each single and important dam project in order to ascertain these factors. The principle of indirect load transmission is supposed to give practical results.

Fashioning a rather short test prism of the foundation material and pouring a similar prism of concrete with it monolithically (see Fig. 14) a test could be made that would duplicate exactly the conditions prevailing in the actual foundation. The samples would be tested in a vertical position, with horizontal forces, so that the bending moment in the plane between the two samples is zero and the shearing stress is maximum. Furthermore, the bond area between the two prisms could be subjected to water pressure simulating an uplift force. With such a testing arrangement the influence on the shearing strength between the dam and the foundation of all contributing factors (such as quality of rock and concrete, uplift and water-tightness, age of concrete, roughening of the bond area, etc.), could be determined

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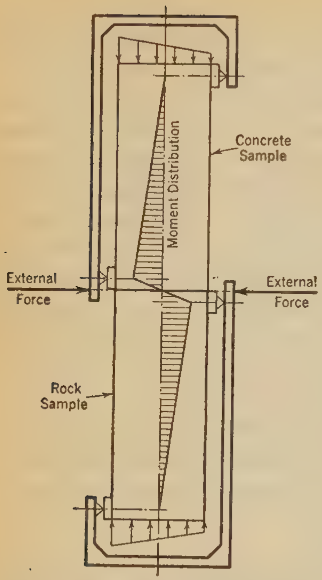

FIG. 14.—TESTING APPARATUS FOR DETERMINING SHEAR STRENGTH BETWEEN ROCK AND CONCRETE.

Concrete is a brittle material and, as such, no stress release is possible at overstressed places as would occur in the case of ductile materials. Consequently, the greatest shearing stress that develops in the dam should be taken for the determination of a shear safety factor. Equations (2), (6), and (13) do not comply with this requirement, but are based on the use of the total shearing force assumed to be distributed equally over the area. It would seem more logical to define this safety factor as the ratio of shear stress at failure to the maximum shear stress in the dam under the most severe condition of loading.

With the author's assumptions (namely, a triangular profile having a vertical water surface, horizontal base, and water at the apex level), the maximum normal and shearing stresses occur at the toe of the base. Equation (8) is also valid for the case of uplift considered; that is, with pressure decreasing from a maximum at the heel to zero at the toe of the base. In the 62.5-lb units used by Mr. Henny, the maximum shearing stress has a value of $\frac{H}{n}$, being maximum at the toe and decreasing uniformly to zero at the heel. The shear safety factor, therefore, should be defined as:

$$Q = \frac{s_1 + K q}{H} n = \frac{s_1 n}{H} + \frac{K}{n} \quad \dots\dots\dots\dots\dots\dots (26)$$

and, similarly, Equation (13) shows decreased factors of shear safety for increasing heights of the dam of constant relative width. It can also be shown that this factor is smallest at the toe of the base.

The author makes the implicit assumption that shear failure takes place simultaneously on the entire base of the dam. In the light of Equation (26), shear failure would begin at the heel and spread progressively toward the water face.

With values of 0.70 for K and 920 for s_1, Equation (26) gives, for the case of a dam with a relative base width of 0.707 H and for heights varying from 200 to 700 ft, the following factors of safety: 4.24, 3.16, 2.61, 2.29, 2.07, and 1.92 instead of 7.49, 5.32, 4.24, 3.59, 3.11, and 2.85 as given in Table 8, Column (3), Items Nos. 10 to 15.

The author's remark that the maximum shear stress will always be less than twice the average and that this range is fully covered by such assumptions as were made in his paper, is somewhat vague. Obviously, the derivation of Equation (26) involves no assumptions regarding the distribution of stresses and strength, and for this reason the writer decidedly prefers such a treatment.

Mr. Henny states that the direct effect of drainage on uplift pressure is difficult to determine; that drains are likely to close and become clogged; and, that for these reasons conservatism in design dictates that no dependence should be placed on the perpetual reduction of uplift pressure by drains. The design of the Italian dam, Suviana,[46] is a good example of the caution taken to avoid clogging of drains. This dam has drain holes about 1 ft in diameter, spaced 20 ft apart. O. Hoffmann has shown the effect of drain holes upon the uplift and gives the following formula,[47]

$$\frac{H_1}{H} = \cfrac{1}{1 + \cfrac{\pi Y}{\cfrac{D \ln D}{\pi R}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

in which, H_1 = uplift pressure at the drain; Y = distance of drain from water face; $2D$ = center distance of the drains; and, R = radius of drain pipe or hole. According to Mr. Hoffmann the uplift decreases from H at the water face to H_1 at the drain and, from there, uniformly to zero at the toe.

The relative area subject to uplift as proposed in Table 6 varies with the heel loading, but does not take into account different qualities of the foundation. The German specifications[48] prescribe the following percentages of area subject to uplift: Good natural conditions of foundation, 20%; average natural conditions, 30%; and less favorable natural conditions, 40 per cent.

It seems to the writer that for estimating the uplift in each particular case the judgment of an experienced engineer and geologist is still the safest and the natural way. In a remarkable paper, M. Lugeon[49] treats of the interrelation of dam design and geology and shows convincingly by means of several failures of dam projects that the co-operation of the engineer and the geologist in establishing the basis of the design is indispensable.

S. H. WOODARD,[50] M. AM. SOC. C. E. (by letter).—For some time the writer has been impressed by the accumulation of evidence that, whatever the cause seemed to be—compression, or shear, or tension—all failures of concrete, mortar, and other like substances are due to tension. The usual failure of a concrete specimen subject to compression is nicely illustrated by Fig. 3 of the paper, which shows the typical conical break. The writer has been led to suspect that the conical breaks may be attributed to the restraining or hooping effect of the friction between the two heads of the machine and the two ends of the cylinder. To test this a large number of cylinders and prisms at the laboratories of the Alabama Power Company were broken in a compression machine having both ends of the cylinders or prisms well lubricated. This was done in various ways, one of the most satisfactory being the use of blotting paper impregnated with heavy grease inserted between the ends of the specimen and the heads of the machine. The specimens cracked vertically from end to end, forming small columns. The results of one set of such tests are given in Table 10.

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[46] "Impianti idroelettrica nella regione appeninica Tosca-Emiliana, alta valle del Reno e Limatra," *Energia Elettrica*, 1925.

[47] "Permeazioni d'acqua e lor effeti nei muri de ritenuta," O. Hoffmann, Milano, 1928.

[48] "Anleitung für den Entwurf, Bau, and Betrieb von Talsperren," Berlin, 1930.

[49] "Barrages et Géologie," M. Lugeon, Lausanne, 1933.

[50] Cons. Engr., New York, N. Y.

TABLE 10.—TESTS OF TENSILE AND COMPRESSION SAMPLES OF PLAIN CONCRETE

Batch No. (1)	Average tensile strength, in pounds per square inch (2)	SPECIMEN		GREASED ENDS				PLAIN ENDS		
		Type (3)	Dimensions, in inches (4)	Compression strength, C, in pounds per square inch* (5)	Type of break† (6)	$\frac{t}{c} = n$		Compression strength, K, in pounds per square inch (9)	Type of break† (10)	$\frac{K}{C}$ (11)
						Each item (7)	Mean (8)			
				(a) 1 : 3 MORTAR SPECIMENS						
2	443	Cylinders	2 by 8	2 480	Col	0.178	0.148	3 430	Con	1.40
2	443	Prisms	4 by 4 by 16	3 750	Col	0.118		3 930	Pyr	1.05
4	419	Cylinders	2 by 6	2 944	Col	0.143	0.145	4 237	Con	1.44
4	419	Prisms	4 by 4 by 8	2 867	Col	0.147		4 105	Pyr	1.40
6	436	Cylinders	2 by 3	3 185	Col	0.138	0.159	3 892	Con	1.20
6	436	Prisms	4 by 4 by 6	2 420	Col	0.180		4 235	Pyr	1.75
8	459	Cylinders	2 by 2	3 603	Col	0.126	0.142	4 014	Con	1.11
8	459	Cubes	4 by 4 by 4	2 880	Col	0.159		4 045	Pyr	1.55
				(b) NEAT CEMENT SPECIMENS						
1	777	Cylinders	2 by 8	9 087	Col	0.086	0.098	12 013	Con	1.20
1	777	Prisms	4 by 4 by 16	6 170	Col	0.110		6 935	Pyr	1.12
3	678	Cylinders	2 by 4	6 300	Col	0.123	0.117	11 187	Con	1.78
3	678	Prisms	4 by 4 by 8	6 020	Col	0.112		7 745	Pyr	1.28
5	634	Cylinders	2 by 3	6 483	Col	0.098	0.097	11 667	Con	1.80
5	634	Prisms	4 by 4 by 6	6 584	Col	0.096		10 220	Pyr	1.56
7	680	Cylinders	2 by 2	6 122	Col	0.111	0.125	11 160	Con	1.83
7	680	Cubes	4 by 4 by 4	4 882	Col	0.138		9 662	Pyr	1.95

* All specimens cracked vertically, forming small columns. † "Col" denotes a columnar break; "Con" denotes a conical break; and "Pyr" denotes a pyramidal break.

A set of 180 cylinders and prisms was made for this test, which were from 2 to 6 in. in diameter, with lengths varying from one to four times their diameter. One-half the samples (that is, 90) were made of neat cement mortar (Table 10 (*b*)), and 90 were made of 1 part cement and 3 parts of standard Ottawa sand (Table 10 (*a*)). From each batch of mortar a set of nine tensile briquettes was moulded at the same time that the compression specimens were made. As there were ten batches there were 90 control tensile briquettes and 180 compression samples.

All specimens were broken at the end of 28 days. One-half of all the compression specimens were broken in the ordinary way without greasing the ends, and the remaining 90 (45 neat and 45 mortar specimens) were broken with greased tops and bottoms. All the latter resulted in columnar breaks and all the former in the usual cone or pyramid at the two ends. These results suggest a tentative law that, normal to the direction of every compressive stress, there is a tensile stress, and that the ratio between the magnitude of the tensile and compression stresses will be a constant factor characteristic of the material. Thus, if p equals a compressive stress, in pounds per square inch, and f equals resulting tensile stress, in pounds per square inch,

$$f = -np \qquad (28)$$

From Table 10(a), it appears that the value of n for those particular samples of 1:3 mortar was between 0.14 and 0.159, while the value for the stronger neat cement mortar (Table 10(b)) was between 0.097 and 0.125.

All materials subject to shortening of one axis by simple compression are known to swell or to elongate in directions normal to that axis the amount represented by Poisson's ratio, and it may be expected that elongation would be accompanied by tension. In Equation (28), n is a ratio of stresses and, therefore, is not Poisson's, which is a ratio of strains; but if the moduli of elasticity are constant the values of the two might be the same. The values of n in Table 10 are near enough to the known values of Poisson's ratio for mortar and concrete to be at least suggestive. As is usual with tensile breaks, individual briquettes broke at tensile values 10% greater and less than the averages shown in Column (2), Table 10, and it is interesting to note that if values of n equal 0.148 for 1:3 mortar and 0.109 for neat cement mortar had been used in Equation (28) to determine the tensile strength from the results of the compression tests with greased ends, the resulting determination of tensile strengths would have been as close to the average of the tensile breaks as were the individual tensile breaks themselves.

The writer is rather diffident about reporting these experiments because the results are incomplete and somewhat erratic. The series was designed as a pilot set of experiments to furnish a guide for a much more elaborate series which had to be abandoned. Further research is wanted to settle, definitely, the question of whether the greased bearing method of testing compression samples does not give a truer determination of the strength of the material under compression than the method now commonly used. Further research would also be required to establish, thoroughly, any such law as is tentatively suggested herein.

A mental picture of the physical operations of such a law may be formed by considering the materials under compression to be made up of particles that are connected by an elastic tensile strength (which may be conceived to be cementation, attraction, adherence, or any elastic holding property) and separated or prevented from nearer approach to each other by a relatively great resistance which may or may not have some elastic properties. These particles may be conceived as anything from Bohr atoms to grains of sand. If they are packed in an amorphous state into any given space, the transfer of force from one side of that space to the other would seem to be as if by tetrahedral frames, the junction points being at the centers of the particles and the members having definite tensile properties and relatively extremely great compressive strength. In such a structure compression in any direction means tension normal to that direction. This may not be a true or complete picture of what happens, but it does illustrate a possibility that would explain the relationship of compressive and tensile stresses.

Referring now to the set of experiments, summarized in Table 3 and Fig. 2, a set of 6 by 12-in. prisms was cast of neat cement mortar, except for a ¾-in. band of mortar made of 1 part cement to 3 parts of standard Ottawa sand. This was done by setting the wooden mould in an inclined position so that the desired shear band would be horizontal. The mould was filled with neat cement mortar up to the under side of the desired shear band and immediately the ¾-in. layer of 1:3 mortar was placed and the remainder of the volume filled with neat cement mortar. The wooden moulds were especially designed and built in three parts, separated at the planes of the two sides of the ¾-in. shear band to permit this to be done without distortion. When the moulds were removed there was no indication that the shear band had been distorted. From each batch of 1:3 mortar which was used to make the shear bands a control set of six tensile briquettes was made, except that for Specimens Nos. 18 and 20 only two each were made. No control samples were made for the neat cement mortar, which is to be regretted.

The fundamental data of Table 11 are the same as in Table 3, with the addition of the breaking strength of the control tensile briquettes. In Column (5), $p = \frac{P}{A}$, of course, is found by dividing Column (3) by Column (4). Items Nos. 15 and 18 (Column (5)) were computed by the formula,

$$p = \frac{P}{2\,a \sin \alpha \cos \beta} \quad \text{..........................} (29)$$

in which, a = the area of the shear plane and β = the angle between the direction of the resultant and that of the load.

TABLE 11.—Tests of Stress on Diagonal Shear Planes

Specimen No. (1)	cot α (2)	Total compression, P, in pounds (3)	Area, A, in square inches (4)	Values of P, in pounds per square inch (5)	Values of f_1, in pounds per square inch (6)	Tensile strength, t (1:3 mortar), in pounds per square inch (7)
1	0.40	145 650	36	4 046	—218	—420
2	0.40	156 270	36	4 341	—240	—338
3	0.80	138 850	36	3 857	—350	—420
4	0.80	141 070	36	3 911	—354	—338
5	0.80	141 170	36	3 921	—355	—312
6	1.20	115 170	36	3 199	—356	—420
7	1.20	95 210	36	2 645	—297	—338
8	1.20	98 320	36	2 731	—305	—312
9	1.75	86 340	36	2 395	—302	—420
10	1.75	99 520	36	2 765	—348	—338
11	1.75	90 300	36	2 508	—316	—312
12	2.33	99 700	36	2 770	—368	—420
13	2.33	91 350	36	2 537	—338	—338
14	2.33	81 880	36	2 275	—303	—312
15	7.95	50 820	..	2 812	—401	—420
16	7.95	50 600	..	2 800	—397	—338
17	7.95	49 090	..	2 720	—386	—312
18	5.15	57 610	..	2 120	—301	—221
19	5.15	66 670	..	2 245	—348	—305
20	5.15	78 480	..	2 880	—407	—277

When making the tests, as soon as the beam of the testing machine dropped, indicating that a specimen had failed, a tracing was made of the cracks showing on the sides of the specimen (see Fig. 15). A line of thrust is indicated on Specimens Nos. 16 and 18. On the other specimens (Nos. 1 to 14, inclusive), the line of thrust, of course, is normal to the bed of the testing machine. The exact position of the line of thrust in Specimens Nos. 15 to 20, inclusive, can only be approximated. A change of position would change, somewhat, the values of f_1 given in Table 11, Column (6), for Specimens Nos. 15 to 20.

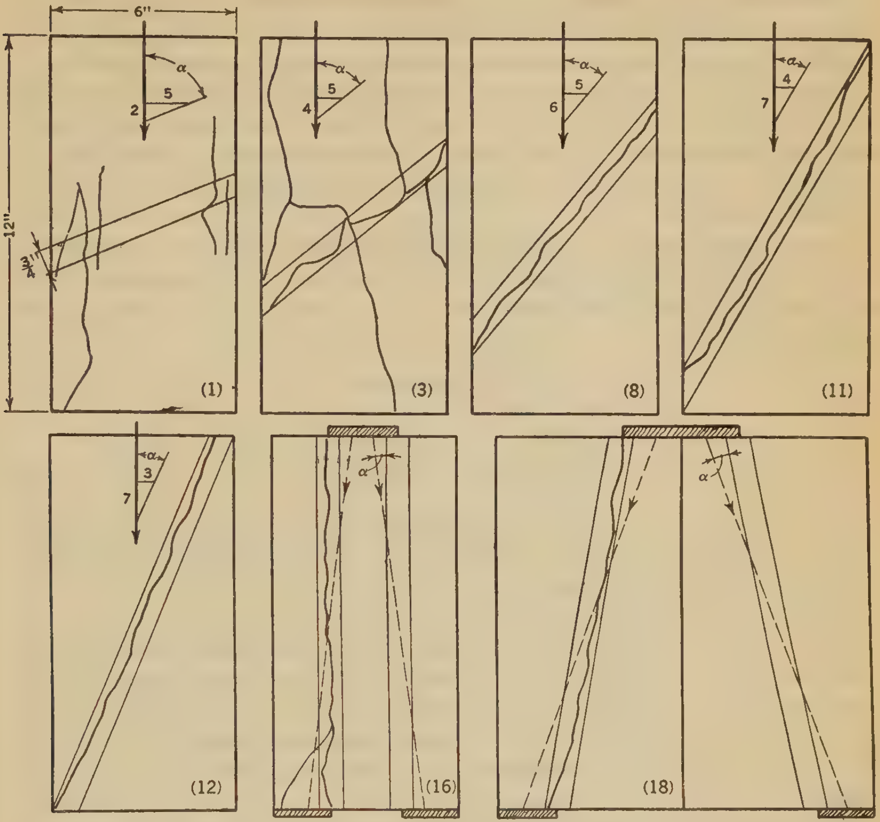

FIG. 15.

Column (6), Table 11, shows the computed tensile stress across the shear planes from the formula,

$$f_1 = -np \cos \alpha \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which, α is the angle between the shear plane and the direction of the compressive forces; p, the compressive stress; and n, as before, a constant which is a characteristic of the material and is the ratio between the tensile stress (set up by the compressive stress normal to the direction of compression) and the compressive stress. For computing Column (6), the value of n is taken to be 0.145, which is a fair average of the values of n for 1:3 mortar shown in Table 10(a). Of course, as this is a factor assumed to be characteristic of the material and as six different batches of 1:3 mortar were used, it is probable that there were six values of n differing slightly among themselves.

The values of f_1 for all specimens that have clear breaks in the shear band are close enough to the values of t to make it seem probable that the formula is at least approximate. If this is so, f_1 is the component of the stress, f, normal to the direction of compression when that stress is resolved normal to the shear plane.

If there is a tensile stress across the shear plane which is a function of the principal compressive stress then it is a function of the components of the principal stress. While this relation is not a simple one (as the author points out), the formulas,

$$f_1 = 0.35q - 0.55s \dots\dots\dots\dots (31)$$

or,

$$f_1 = 035p \sin^2 \alpha - 0.55p \sin \alpha \cos \alpha \dots\dots\dots\dots (32)$$

will give values of $- f_1$ which are consistent with the results of the tests of Table 11. In Equations (31) and (32), q = the unit compression on the shear plane, and s is the unit shear.

While the values of f_1 computed from Equations (30) and (31) are consistent with the breaks, they are not consistent with each other because, when α is greater than about 68°, Equation (30) gives negative values and Equation (31) gives positive values. As the specimens having shear planes at that angle failed in the neat cement mortar (Specimens Nos. 1, 2, 3, 6, and 9, in Column (7) Table 10), what would have happened in the shear band is not known.

If the force, P, is resolved into two components, Q, normal to the shear planes (producing the compressive stress, q, on the shear plane), and V, in a direction of the shear plane (producing a compressive stress, v, on a plane normal to the shear plane), the equation,

$$f_1 = - \frac{2\,nq}{3} - n\,v \dots\dots\dots\dots (33)$$

or

$$f_1 = - \frac{2\,np \sin^2 \alpha}{3} - np \cos^2 \alpha \dots\dots\dots\dots (34)$$

will also give values of f_1 that will be consistent with the results of the breaks of the specimens in Table 11.

However, the experiments are too crude and limited in number to establish any formula and there is uncertainty as to the effect of the probable higher modulus of elasticity for the neat cement portion of the prisms. It is hoped that they may incite some one to frame other experiments which will give more conclusive results.

The experiments on the force of water in the pores of mortar referred to in Table 7 are probably not conclusive. They should be repeated, with a determination of the time element. However, in gravity dams, the danger is principally from water pressure in horizontal construction joints in which the cross-section of pores may be many times the cross-section of pores in the adjacent concrete.

With reference to construction joints between the layers of successive concrete placing in gravity dams, the writer has adopted the practice of sloping them downward from the down-stream face to the up-stream face of the dam; thus, they are not in the critical section, which is horizontal. It is found that concrete can be placed on a slope of 1 on 10 as conveniently as horizontal.

A. Floris,[51] Esq. (by letter).—In this valuable paper the author proposes the abandonment of certain antiquated rules and conceptions used in the design of dams, which are contrary to present knowledge on the subject. Similar suggestions were made by the writer in 1929.[52]

The subject of the ultimate strength of gravity dams leads, obviously, to the problem of rupture in brittle materials subjected to external forces. It appears that Mohr's theory of failure is in better agreement with facts than Coulomb's theory,[53] which is preferred by the author.

Engineers use working stresses, and not conditions of stress near the breaking point. It is true that allowable stresses are determined in relation to the ultimate strength of the material; but in dams other factors must be considered simultaneously. By assuming the validity of the rule of the trapezoid, all other stresses follow from this assumption. Usually, the principal normal stress governs the design and not the principal shearing stress. According to theory the greatest shearing stress in a two-dimensional state of stress is equal to one-half the principal compressive normal stress, when the principal tensile normal stress vanishes. Therefore, if the compressive stresses are kept within reasonable limits, and tensile stresses are not allowed, it is evident that the danger of failure within the dam itself will be remote. In fact, all recorded failures of gravity dams have occurred at the base where two different materials come into contact.

In view of the importance of the subject to the safety of gravity dams, N. Kelen has made tests on stone and concrete models for the purpose of determining the bond, shearing, and frictional resistances of the models at their base.[54] In these unique tests an additional normal force was acting simultaneously, and a linear relationship was found between the shearing and normal stresses. In addition, when the bond vanished at the base of the models, the frictional resistance increased appreciably. A great decrease in shearing resistance was also observed, caused by the action of uplift at the base of the models.

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[51] Los Angeles, Calif.

[52] "Sliding Factor in Dams," by A. Floris, *Western Construction News*, March 10, 1929, p. 141.

[53] "Welche Umstaende bedingen die Elastizitaetsgrenzen und den Bruch eines Materials?" Abhandlungen aus dem Gebiete der technischen Mechanik, von Otto Mohr, Berlin, 1914, p. 192.

[54] "Versuche zur Bestimmung des tangentialen Sohlenwiderstandes von Gewichtsstaumauern," von N. Kelen, Berlin, 1933.

The question of uplift in gravity dams has been solved theoretically by P. Fillunger and does not involve great uncertainties, as it is often claimed.[55] Unfortunately, his findings have been ignored by engineers in practice. To make use of this theory it is necessary only to determine certain constants experimentally.

According to this theory, when the water penetrates the pores of the dam it exerts an hydrostatic uplift and a liquid friction, acting in the direction of the steady flow of water within the dam. The hydrostatic uplift decreases the specific weight of the structure and the liquid friction increases its overturning moment so that both forces cannot be ignored.

Different conditions may prevail at the base of the dam where, in order to avoid tension, the porosity of the rock must at least be equal to, or less than, the porosity of the dam. This can be accomplished by artificial means, such as by grouting.

JOSEPH JACOBS,[56] M. AM. SOC. C. E. (by letter).—The profession is indebted to the author for an exceptionally clear analysis of the elements of stability in concrete gravity dams. In the design of any important engineering structure no reasonable test by which its safety may be judged, should be ignored.

There are several criteria by which the base width of concrete gravity dams may be determined for any height. The controlling one is dependent on the variable elements of height of dam, character of foundation, character of concrete used, and the relative safety factors and allowable unit stresses deemed essential, in the particular case under consideration. In general, that criterion is most vital which compels the greatest base width of dam and on that assumption, in conjunction with certain analytical considerations hereinafter referred to, it would seem to the writer that the criterion of shear is not clearly entitled to the high status assigned to it by the author; nor is it believed that it should take absolute precedence over the criterion of sliding as a test for stability.

Regardless of what the sliding factor may be the keying of the base of the dam into the foundation, of course, will prevent continuous sliding if adequate horizontal, compressive resistance is provided. Such anchorage into the foundation is altogether desirable but, on account of the shrinkage of fresh concrete, it may not, under full load, prevent incipient sliding if the base width of the dam is less than that required for safety against sliding. This latter condition not only imposes a greater unit shearing stress, and a greater vertical compressive stress at the toe of dam, than would otherwise obtain, but it also introduces the element of horizontal compression at the toe, to resist which an adequate depth of anchorage into the foundation rock must be provided. Under certain conditions, particularly for high dams and for poor foundation material, these considerations may make it quite unwise and even uneconomical to ignore sliding.

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[55] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 200; and "Auftrieb und Unterdruck in Staumauern," von P. Fillunger, *Transactions*, Second World Power Conference, Berlin, 1930, Vol. IX, p. 323, and summary in English, p. 328.

[56] Cons. Engr. (Jacobs & Ober), Seattle, Wash.

There are fewer uncertainties about sliding than about shear. For the former, the major uncertainty is the coefficient of friction and this is determinable within reasonable limits for any given case, particularly after the foundation has been exposed. For shear, however, there are several uncertain elements, as follows:

(*a*) The extent to which allowance should be made, if any, for increased shearing strength due to loading normal to the plane of shear (the amount of this increased strength is not yet well known and, as an element in dam design, it has usually been ignored);

(*b*) The relationship of base shear for a vertical section of small unit width (which is the usual basis of computation) to shear strength of the dam as a whole or a considerable portion thereof (which is the more likely method of actual shear failure); and,

(*c*) The distribution and maximum unit intensity of shear on a horizontal section of dam.

According to different authorities, the shear may be distributed uniformly, it may have a straight-line or curved-line variation with maximum intensity at the heel and zero at the toe, or exactly the reverse of that; or, it may vary as a parabola with maximum intensity at the center of the section. It is indeed strange that pure theory and experimentation have not yet determined this matter definitely and acceptably to the Engineering Profession.

For purposes of comparison, the writer has determined the required base widths of gravity dams, in terms of their heights, for varying factors of safety and for various critera, such as overturning, sliding, shear, etc. The author's dam of triangular cross-section with vertical back was used for this comparison; the values of n were reduced to their simplest form; and the analyses were confined to two simple cases, as follows:

Case I.—A dam of triangular cross-section with vertical back; reservoir full; no uplift; no allowance for increased shearing strength due to loading; no allowance for base adhesion in considering overturning.

Case II.—A dam of triangular cross-section with vertical back; reservoir full; allowance is made for uplift amounting to full head at the heel and zero at the toe, applied to a 40% pore area over the base section of the dam. Furthermore, an allowance is made for increased shearing strength due to loading, amounting to 70% of that loading, this being the value used by the author. No allowance is made for base adhesion in considering overturning.

The symbols in Table 12 are defined as follows: h = height of dam, in feet; n = ratio of base width to height of dam; w = weight of 1 cu ft of water = 62.5 lb; a = specific gravity of concrete = 2.4, the concrete weighing 150 lb per cu ft; F = factor of safety; c = unit compressive strength of concrete, at

28 days, in pounds per square foot; s = unit shear strength of concrete, at 28 days, in pounds per square foot (unloaded); and p = loading at heel in terms of full head, hydraulic pressure, wh.

TABLE 12.—BASE WIDTHS OF GRAVITY DAMS IN TERMS OF HEIGHT

Criterion	CASE I	CASE II
Overturning	$n = \sqrt{\frac{F}{2a}} = 0.456\sqrt{F}$	$n = \sqrt{\frac{F}{2a - 0.8}} = 0.50\sqrt{F}$
Sliding	$n = \frac{F}{af} = \frac{F}{2.4f}$	$n = \frac{F}{f(a - 0.4)} = \frac{F}{2f}$
Shear	$n = \frac{Fwh}{2s} = \frac{31.25Fh}{s}$	$n = \frac{whF}{2s + kwh(a - 0.4)} = \frac{31.25hF}{s + 43.75h}$
Loading at toe	$n = \sqrt{\frac{whF}{c}} = \sqrt{\frac{62.5hF}{c}}$	$n = \sqrt{\frac{whF}{c}} = \sqrt{\frac{62.5hF}{c}}$
Loading at heel	$n = \sqrt{\frac{1}{a - p}} = \sqrt{\frac{1}{2.4 - p}}$ For $p = 0$, $n = 0.6455$ For $p = 0.4$, $n = 0.7071$ For $p = 1.0$, $n = 0.8452$	$n = \sqrt{\frac{1}{a - p}} = \sqrt{\frac{1}{2.4 - p}}$ For $p = 0$, $n = 0.6455$ For $p = 0.4$, $n = 0.7071$ For $p = 1.0$, $n = 0.8452$

It will be noted from Table 12 that for overturning and for sliding, the value of n is independent of the height, h. It is of interest to note, also, that for $n = 0.6455$ the compression at toe of dam for reservoir full is the same as compression at heel of dam for reservoir empty. For this base width the limiting height of the dam, as controlled by the allowed unit compressive stress, is 480 ft for an allowed unit stress of 500 lb per sq in.; 576 ft for 600 lb per sq in.; and 672 ft for 700 lb per sq in. Similarly, for $n = 0.8452$, while the limiting heights, for reservoir full, would be very much greater than the preceding values, the actual limiting height would be controlled by compression at the heel for reservoir empty and would be exactly the same as the preceding values.

In other words, the limiting height of a dam of triangular section with vertical back, for reservoir empty, is independent of the base width and equals the allowed unit compressive stress, in pounds per square foot, divided by the weight of 1 cu ft of concrete. It is evident, then, that for high dams a richer and stronger concrete is required than for low dams. For a dam of triangular section and a height of 500 ft, for instance, a concrete would be required that is strong enough to justify an allowed unit stress of 520 lb per sq in., which, for a safety factor of 4, might require a 1:2:4 concrete mix. For a dam 250 ft high, however, the allowed unit stress would need be only 260 lb per sq in., for which a 1:3:5 concrete would suffice.

From Table 12, and covering a considerable range in the value of the variables involved, graphs were prepared to afford a visual concept of the varying requirements as to base width. The graph for Case I is herein shown as Fig. 16. Assuming such reasonably conservative values of the variables as are likely to obtain for any particular case, it is apparent from this diagram that the consideration of sliding will generally demand the highest values of n, the principal exceptions thereto being considerations of shear and of toe compression for extremely high dams. In respect of sliding for trapezoidal sections the base width would be decreased by an amount equal to the top width of dam and, for normal ratios of top width to bottom width, the preceding statement would still be essentially correct although the disparities between n for sliding and n for other criteria would be less than those for dams of triangular section, as shown in Fig. 16. For Case II practically the same general condition obtains as for Case I. Despite the fact that the element of shear should always be carefully considered, particularly for high dams, it would seem to the writer, in the light of the foregoing indications, that the criterion of sliding should not be discarded in the design of gravity dams.

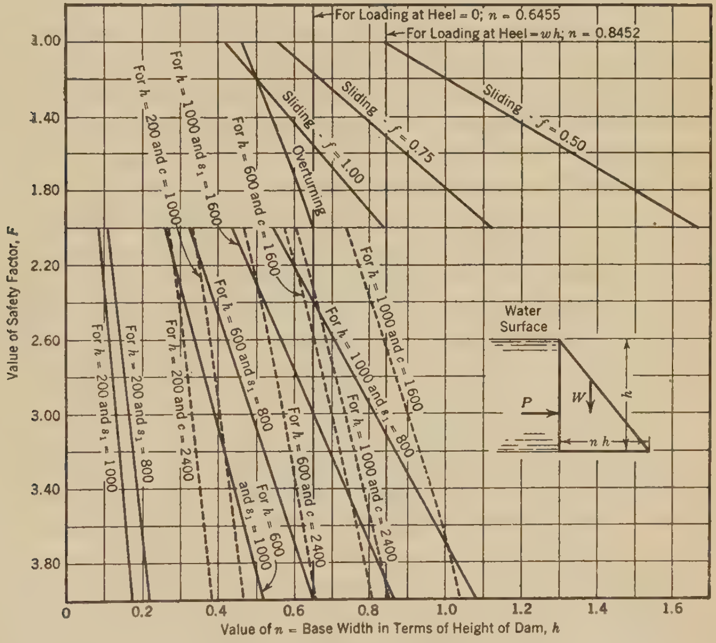

FIG. 16.—BASE WIDTHS OF STRAIGHT CONCRETE GRAVITY DAMS OF TRIANGULAR CROSS-SECTION FOR VARYING SAFETY FACTORS AGAINST OVERTURNING, SLIDING, SHEAR, ETC.

In conclusion, the writer desires to make one inquiry concerning the Woodard experiments, on possible water pressure in the pores of cured mortar specimens, to which the author refers. In reaching his own conclusions from these experiments, particularly the specific conclusions as to quantitative values of these internal water pressures, did the author give full consideration to the fact that a non-uniform distribution of the pores throughout the mass would mean a pore area on some sections greatly in excess of the mean pore area of all sections? This condition, in turn, creates a section subjected to a greater water pressure than the average section and, at the same time, has a smaller resisting area of solid material. Such a non-uniform distribution is almost certain to prevail and, for identical exterior loading, some sections are subjected to unit stresses probably double that of other sections. The writer also wonders if the effect of Poisson's ratio on unit stress was given consideration. The varying values of Poisson's ratio and of the modulus of elasticity, for sand-cement mortars of varying consistency, are not known accurately but, using conservative values for these factors, it would seem, by increasing the lateral pressure of the first three items of Table 7 about 50%, that the induced Poisson ratio, tensile stress would be sufficient to break the specimen, entirely independent of any stress caused by internal water pressure. It would seem, also, that consideration of Poisson's ratio would tend to lessen the disparities shown in Column (7), Table 7.

ROBERT E. GLOVER,[57] ESQ. (by letter).—Under the heading, "Shearing Strength," the author discusses the influence of the stress normal to the shearing plane upon the shearing resistance, and cites tests to show that the shearing resistance of the material is increased by a compression normal to the shearing plane. Punching shear tests made by Professor A. N. Talbot, and shear tests by Mr. Woodard, are quoted.

It seems apparent that the purpose of the punching shear tests, and the tests on specimens of Type (*b*), Fig. 2, where the mortar zones were disposed vertically in the specimen, was to permit a direct evaluation of the shearing strength of concrete when the stress normal to the shearing plane is zero. The conditions necessary for the maintenance of equilibrium require that when a body is subjected to stress in one direction certain intensities of shear and normal stress must exist in all other directions, and since it is known that a pure shear stress is accompanied by compressive and tensile stresses of equal intensity on planes disposed at an angle of 45° with respect to the planes of maximum shear, it becomes pertinent to inquire whether the weakness of concrete in tension would not render such a direct determination impossible.

For a two-dimensional case the question may be attacked as follows: The stress condition in such a solid may be specified completely in terms of the two principal stresses and the inclination of the principal axes to an arbitrary plane of reference. If the material of the body is isotropic and the question is one which concerns the causes of failure, only the magnitudes of the principal stresses are significant. It will be found that if there is to be a plane having a zero normal stress, it would be necessary for one of the principal stresses to be compressive and the other tensile. Under these conditions, it can be shown that, if the law of failure by sliding is as stated in Equation (3), the plane of least resistance is independent of the magnitudes of the principal stresses and is identical with that shown in Fig. 1. In order to obtain a shear failure with no stress normal to the plane of shear it would be necessary to make the plane of zero normal stress coincide with the plane of least resistance. It can be shown readily that an experiment made in this manner would probably be terminated by a tension failure before the shearing stresses could be raised high enough to cause a shear failure.

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[57] Engr., U. S. Bureau of Reclamation, Denver, Colo.

Consider the 1:3:6 concrete described by the author immediately after Table 1, with $k = 0.70$ and $\delta = 62° 30'$. In order to develop a shearing stress of 783 lb per sq in. on the plane of least resistance, with no stress normal to the plane of shear, would require principal stresses of 1 500 lb per sq in. compression, and 407 lb per sq in. tension, respectively. Since the tensile strength of such a concrete would probably be about 300 lb per sq in., it follows that a direct determination with such a concrete may be impossible.

The author records evidence of normal stress in the punching shear tests and notes that the results of the tests on the specimens with the mortar zones "do not agree with the theory of Coulomb." The writer does not believe that the assumption of a uniform shear distribution in the case of the punching shear tests or of the tests on the specimens shown in Fig. 2 (*b*), is warranted; and until the possibility of determining the value of s_1 in Equation (3) by such tests, is demonstrated, will hold to the conclusion that they are inadequate for this purpose. It may be remarked, however, that as long as the designers of gravity dams adhere to the policy of eliminating tensile stresses from their structures, their purposes will be served if the laws of failure within the region where all the stresses are compressive, are determined.

Charles Terzaghi,[58] M. Am. Soc. C. E. (by letter).—The two most fundamental and controversial issues of Mr. Henny's stimulating paper concern the shear-friction factor and the effect of hydrostatic uplift on stability.

Considering the present-day tendency to construct concrete dams to unusual heights, it is no longer possible to ignore the effect of the shearing strength on the resistance of dams against sliding along the base. For this reason, most of the related experimental investigations suggested by Mr. Henny have already been made largely during the two years, 1932 and 1933. A. Hertwig and his co-workers[59] have made torsion tests under variable axial loading and obtained for the relation between the shearing resistance, s, and the axial load, q (both in pounds per square inch), the approximate formula:

$$s = 430 + 1.13\, q \qquad (35)$$

The specimens consisted of 1:5 concrete with water-cement ratios of 0.37. They were 21 in. high and had a diameter of 11 in.

N. Kelen[60] has determined the shearing resistance along smooth and rough surfaces of contact between concrete and various rocks (sandstone, limestone, and granite) under different surface pressures and has obtained the equation:

$$s = 253 + 0.8\, q \qquad (36)$$

In his experiments the plane of shear had an area of 1 sq ft.

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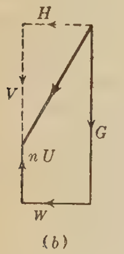
[58] Dr. Ing.; Prof., Technische Hochschule, Vienna, Austria.

[59] "Zur Frage der Bruchsicherheit von Staumauern," von A. Hertwig, A. Ludin, und H. Petermann, *Deutsche Wasserwirtschaft*, 1933, Heft 7.

[60] "Versuche zur Bestimmung des tangentialen Sohlenwiderstandes von Gewichtsstaumauern," von N. Kelen, Berlin, 1933.

In both series of experiments the friction coefficient, k (using the author's nomenclature), appreciably decreased with increasing surface pressure. According to the results of theoretical investigations by A. Brandtzaeg,[61] Assoc. M. Am. Soc. C. E., this phenomenon must be considered a universal property of non-isotropic bodies in general, to which class the concrete also belongs. Hence, the Coulomb formula can merely be considered to be a crude approximation. Friction values computed from the results of the measurement of the angle, $90 - \gamma$, between the planes of shear formed by crushing tests with concrete specimens are utterly unreliable. According to numerous and very accurate data published by M. Ros,[62] the deviations from the theoretical relation between $90 - \gamma$ and the corresponding angle of internal friction for concrete and rocks are very considerable and erratic.

The second outstanding feature of Mr. Henny's paper is the fact that it calls attention to the effect of hydraulic uplift on the stability of dams. In 1933, the writer was compelled to clarify his own ideas concerning this subject. Fig. 17 represents a section through a concrete gravity dam. It is desired to determine the vertical component of the force that acts on the base of the dam.



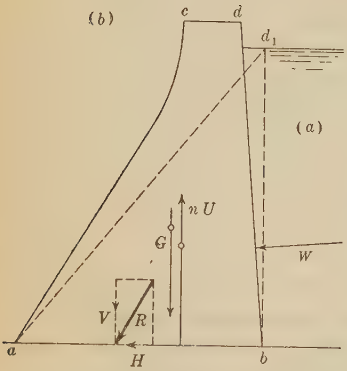


FIG. 17

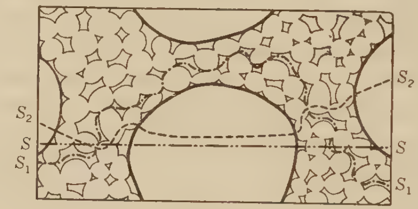


Fig. 18

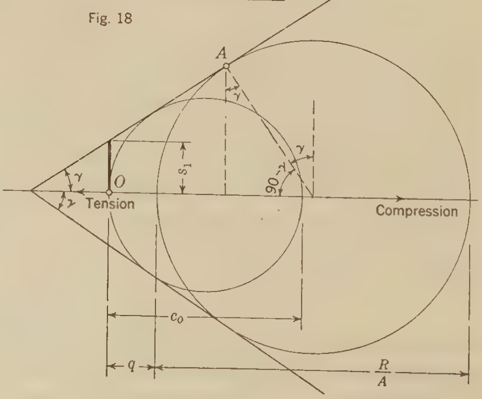


FIG. 19

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[61] "Failure of a Material Composed of Non-Isotropic Elements," by A. Brandtzaeg, Königlich-norwegische wissenschaftliche Gesellschaft, Trondhjem, 1927. Compare, also, *Bulletin No. 185*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[62] "Versuche zur Klärung der Frage der Bruchgefahr, II, Nichtmetallische Stoffe," von M. Roš und A. Eichinger, Eidgenössische Materialprüfungsanstalt an der E. T. H. in Zurich, June, 1928.

Let A = the area of the cross-section of the dam;

G = its weight in a saturated condition;

μ = the fraction of the total volume of a dam occupied by voids (0.12 to 0.15);

n = the unknown effective uplift area (fraction of the total area of the base);

$n\ U$ = the hydrostatic uplift which acts on the effective uplift area within the base; and,

V = the vertical component of the forces which acts on the solid substance within the base.

In Fig. 17 the value, U, is represented by the area of the triangle, $a\ b\ d_1$. The ordinates of $a\ d_1$ indicate the height to which the water would rise in a stand-pipe with its lower end located within the base. The line, $a\ d_1$, is usually assumed to be straight, as shown. Hence, if n is known, the value, $n\ U$, is also determined and the unknown force, V, is obtained by means of a simple polygon of forces (see Fig. 17 (*b*)). Therefore, the problem consists merely in determining the value, n.

Fig. 18 shows an enlarged detail of a vertical section through the concrete immediately above the base of the dam. If a perfectly plane, horizontal cut, $S\ S$, is made through the concrete, immediately above the base of the dam, the pore space for this cut will, for statistical reasons, be equal to the volume of voids, μ (0.12 to 0.15), of the concrete (see Fig. 18). However, a shear failure never occurs along a perfectly smooth plane. The plane of rupture is always rough, and for a rough section through the concrete the pore space per unit of area can have any value between the two extreme limits, $n_{\min} < \mu$ (rough section, $S_2\ S_2$) and $n_{\max} > \mu$ (rough section, $S_1\ S_1$). Hence, the uplift, $n\ U$, can also have any value between $U n_{\min}$ and $U n_{\max}$ depending on the details of the rough section; yet, for the purpose of evaluating the safety of the dam against shear along the base, only one of these values can be correct. It is the value, n_w, of the porosity of that particular rough section which interconnects within the concrete—the spots of least resistance—that is, where the shearing actually occurs.

The experimental method used by the writer for determining the porosity (effective uplift area), n_w, for sections of least resistance, is based on the well-known rupture diagram of Mohr for concrete. Originally, Mohr's diagram merely represented a procedure for plotting test results graphically. As a result of Professor Brandtzaeg's investigations[61] it is known that the diagram in the form obtained for concrete and for rocks, expresses the very essence of the resistance of solid, non-isotropic bodies with internal friction. Fig. 19 shows such a rupture diagram for a cylindrical specimen of concrete with a cross-sectional area, A, the failure of which occurred under the influence of the following forces: An external pressure, q, per unit of area, acting everywhere on the solid substance of the specimen, perpendicular to its outer surface, and an axial load, R. Point O (Fig. 19), is called the "pole" of the diagram. The abscissas of the two points of intersection between any one of the circles shown in the diagram and the horizontal axis represents one pair of principal stresses required to produce failure. The co-ordinates of the points of contact between the circle and the inclined tangent (as, for example, Point *A*, Fig. 19), determine the shearing stress (ordinate) and the normal stress (abscissas) which act in the plane of rupture (shearing plane), and $90 - \gamma$ is the angle formed between the two co-ordinated planes of shear. In practice, for reasons explained by Professor Brandtzaeg[61] the envelopes of the circles are always curved, becoming flatter with increasing distance from the pole. The variable slope of these curves represents the variable coefficient of internal friction. For practical purposes one usually replaces the variable coefficient by an average coefficient ($\tan \gamma$). The effect of this simplifying assumption is that the envelope of the circles becomes a straight tangent forming an angle, γ, with the horizontal axis, as shown in Fig. 19.

Let s_1 = the shearing resistance along a shear plane on which the normal stress is equal to zero; and γ = the average angle of internal friction. Using these symbols, the axial load, *R* (Fig. 19), required for breaking the specimen, is:

$$R = \frac{2\,A}{1 - \sin\gamma}\,(s_1 \cos\gamma + q \sin\gamma) \quad \ldots\ldots\ldots\ldots\ldots (37)$$

In Equation (37), let $q = 0$ and $A = 1$; then the simple compressive strength is:

$$c_0 = \frac{2\,s_1 \cos\gamma}{1 - \sin\gamma} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

If the breaking load, *R*, should be determined for a specimen completely immersed in a liquid under a hydrostatic pressure, *p*, a distinction must be made between two cases, as follows: (*a*) The specimen is entirely surrounded by a water-tight membrane (protected specimen); or, (*b*) the voids of the specimen communicate freely with the water under pressure (free specimen).

In Case (*a*) the entire hydrostatic pressure acts on the solid substance of the specimen and the value, *R*, is obtained from Equation (37) by substituting $q = p$. In Case (*b*) the following fact must be considered: Since the external pressure, *p*, and the internal stresses of the body balance each other, the normal stress acting on any section across the body must also be equal to *p*. Due to the fact that every section is porous and that the pores communicate freely with the external water, one part, $n\,p$, of the normal stress, is taken up by the water, wherein *n* denotes the pore space per unit of area of the section. Hence, the normal stress produced by the liquid in the solid substance of the specimen merely equals $p - n\,p = p\,(1 - n)$. As mentioned previously the value of *n* ranges between the extreme limits, $n_{\min}$ and $n_{\max}$; and rupture can occur only along rough sections representing planes of greatest weakness. The porosity of such planes has been designated by n_w. For a section of this type, the normal stress acting in the solid substance is $p\,(1 - n_w)$ per unit of area. According to the preceding comments, this stress is equal to the normal stress produced by an external pressure, $q = p\,(1 - n_w)$, acting on a protected specimen. Substituting this value in Equation (37):

$$R = \frac{2\,A}{1 - \sin\gamma}\,[s_1 \cos\gamma + p\,(1 - n_w) \sin\gamma] = [c_0 + p\,(1 - n_w) \sin\gamma]\,A$$

Whence,

$$n_w = 1 - \left(\frac{R}{A} - c_0\right) \frac{1 - \sin \gamma}{2\, p \sin \gamma} \quad \text{.................} (39)$$

The derivation of Equation (39) is based entirely on elementary rules concerning the resistance of materials. The only premise that transcends the boundaries of these rules is the assumption that the angle, γ, of internal friction is independent of the intensity of the hydrostatic pressure, p, acting in the liquid content of the voids of the concrete. However, since it is known, from the results of numerous tests, that the angles of internal friction of dry and of saturated concrete are identical, it is impossible that a mere increase of the hydrostatic pressure of the water could eliminate internal friction. Hence, Equation (39) must be considered fundamentally sound.

For any assumed value of p, Equation (39) contains the following unknown quantities: The simple compressive strength, c_0, of the concrete; the breaking load, R; and the angle, γ, of internal friction. Each one of these elements can be determined by test. Therefore, the formula can be used for determining the effective uplift area, n_w.

Related tests were made during the summer of 1933 in the writer's laboratory in Vienna, which yielded the following clean-cut results: The value, n_w, is at least equal to 0.95.[63] This rule holds true even for cement mortar of 1:3, and very likely holds true for neat cement. According to tests made by the writer, the volume of voids of neat cement ranges between less than 0.28 and more than 0.48, depending on the water-cement factor.

The aforementioned results should agree with those of any test made along similar lines, including those of S. H. Woodard, M. Am. Soc. C. E., cited by Mr. Henny. Unfortunately, Mr. Woodard selected a method such that a quantitative interpretation of the test results is practically impossible. Fig. 20 shows a longitudinal section of one of Mr. Woodard's specimens. The water entered the body horizontally and escaped vertically. Due to its flowing through the body along curved lines a complicated state of tri-axial loading was produced excluding the possibility of direct comparison with the state of stress in a briquette under simple tension. Furthermore, in order to compute the tensile stresses produced by the water, the distribution of the hydrostatic pressure over the plane of rupture, *a b*, should be known. This distribution can be very different depending on whether or not the permeability of the specimen was perfectly uniform. When a specimen is moulded a skin is formed with a relative permeability that cannot even be guessed. Nevertheless, the test results do show clearly that pore pressure is exerted over a much larger proportion of the cross-section than that represented by porosity. To that extent Mr. Woodard's tests served their purpose fully.

Dividing the tensile strength of the concrete by the effective unit pore space, n_w, it is recognized that the true tensile strength of the binder must at least be equal to 10 000 lb per sq in. of the true section of rupture. Twenty years ago such a value would have been startling. However, during the decade, 1924-1934, numerous investigations have revealed the fact that the unit tensile strength increases rapidly with decreasing cross-sectional area of the body subject to tension, provided this area is less than approximately 1 sq mm. Thus, for very thin glass threads, values were obtained as great as 1 000 000 lb per sq in.[64] Since the tensile strength of the cement is concentrated at the minute areas of interconnection between the individual grains of cement, scattered all over the plane of "maximum weakness," its high true strength merely represents an analogy to the high tensile strength of thin threads.

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[63] "Die wirksame Flächenporosität des Betons," von K. Terzaghi, *Zeitschrift des österreichischen Ingenieur und Architektenvereines*, 1934, Heft 1-2.

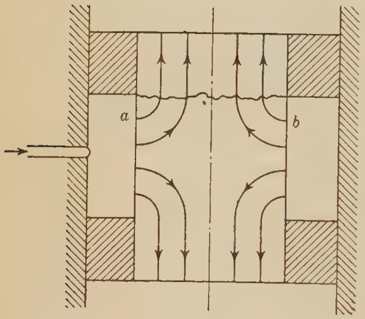

FIG. 20

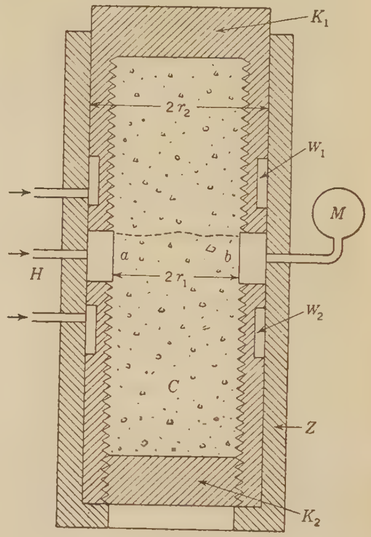

FIG. 21

If the high true tensile strength of the concrete, revealed by the writer's experimental results, should lead some engineers to doubt them, the value, n_w, could be determined directly by means of the apparatus shown in Fig. 21, which represents a cylindrical concrete specimen, C, with its ends fastened into metal caps, K_1 and K_2. The caps are free to slide in the interior of a cylinder, Z; the central part of the specimen is surrounded by an annular space that communicates with a pressure gauge, a, through a pipe line, H, with a source for water under pressure; and the force, P, required to produce failure by tension is assumed to be known from the results of independent tests. By raising the water pressure, p, gradually, the specimen comes under tension until, at a pressure, $p_{\max}$, rupture will occur along a fairly plane, rough surface, a b (Fig. 21). At the instant of rupture the following forces act on the specimen, in the direction of its axis: The hydrostatic pressure, $p_{\max}(r^2_2 - r^2_1)\pi$, acting on the annular rim of the caps, and the pressure, $p_{\max} r^2_1 \pi n_w$, acting on the effective uplift area within the voids of the plane of rupture. Therefore, between P and $p_{\max}$ the following relation exists:

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[64] See, for instance, "Handbuch der physikalischen und technischen Mechanik," von Auerbach und Hort, IV Band, 2 Hälfte, Leipzig, 1931, Seite 19.

$$P = p_{\max} (r^2_2 - r^2_1) \pi + p_{\max} r^2_1 \pi n_w \quad \text{.............} (40)$$

or,

$$n_w = \frac{P}{p_{\max} r^2_1 \pi} - \frac{r^2_2 - r^2_1}{r^2_1} \quad \text{..................} (41)$$

When making these tests, the following precautions should be taken: (1) The specimen should be cured under water until the test begins; (2) the ends of the metal caps should be perfectly water-tight in order to be sure that there is no movement of the water within the voids of the concrete; (3) the specimen for determining the breaking force, P, should be identical in every respect with those for the hydraulic tests; and (4) the axial force should be applied at the ends of the caps, K_1 and K_2. Precaution (4) is necessary, because in both series of tests the distribution of the tensile stresses over the plane of rupture should be the same. Since it is difficult to prevent severe losses of water through the joint between the caps (K_1 and K_2) and the cylinder, Z, it would be advisable to provide the caps with annular grooves, w_1 and w_2. During the tests these grooves should be filled with heavy lubricating oil under a pressure that rises simultaneously with that of the water.

Two points of this discussion should be emphasized in connection with Mr. Henny's paper. The writer recently had an opportunity to collect some personal experiences with high dams in North Africa, and his attention was drawn toward one important source of danger which was not mentioned by Mr. Henny. When constructing a dam on shales or young sandstones the modulus of elasticity of the sub-strata is likely to be considerably less than that of the concrete. In such a case important secondary stresses develop within the masonry, which entirely upset the result of the customary computation of stresses. No softening of the strata beneath the down-stream part of the dam is required for producing this condition. According to the writer's impressions, the failure of the Habra Dam, as well as that of the dam at Bouzey, was essentially due to that primary cause.

The second point concerns the effect of grouting on the distribution of the hydrostatic uplift over the base of the dam. On the basis, both of personal experience and of published data, the writer arrived at the conclusion that the assumption concerning the distribution of the hydrostatic uplift over the base of a dam should be different, depending on whether or not the fissures of the rock are wide enough to be grouted. Repeatedly, the writer could convince himself of the fact that the permeability of rocks with systems of fissures just wide enough to take the grout is several hundred times greater than that of a good concrete, the effective uplift area of which was found to be greater than 0.95. Therefore, the incapacity of a rock to take the grout by no means excludes the danger of hydrostatic uplift. If H denotes the difference in elevation between the water levels on the up-stream and down-stream sides of the dam, H_w, the hydrostatic uplift under the up-stream edge, and H_t, under the down-stream edge, the writer's assumptions are, as follows:

For Class *A*, including hard rocks (granite, solid limestone), with effective grouting below the up-stream edge, $H_w = \frac{2}{3} H$; and $H_t = 0$.

For Class *B*, including hard rock with fissures too narrow to be grouted, or softer rock (schists, shales), with effective grouting below the down-stream edge, $H_w = H$; and $H_t = 0$.

For Class *C*, including softer rock, where no grouting is feasible, $H_w = H$; and $H_t = \frac{1}{2} H$.

For Classes *A* and *C* the corners of the uplift diagrams are rounded off as shown in Fig. 22(*a*) to Fig. 22(*c*). Briefly stated, the arguments on which this classification is based are as follows: If a rock has been grouted successfully, its permeability increases from the heel toward the toe of the dam, and the open fissures below the toe act as a sub-drainage which reduces uplift (Class *A*). On the other hand, if a soft, fissured rock cannot be grouted, its fissures will be narrower below the toe than below the heel, because the pressure exerted by the toe tends to close the fissures. Consequently, below the base of the dam, the permeability of the rock decreases from the heel toward the toe, which, in turn, produces a relatively important hydrostatic uplift in the vicinity of the heel (Class *C*). Class (*B*) represents a state intermediate between Classes (*A*) and (*C*).

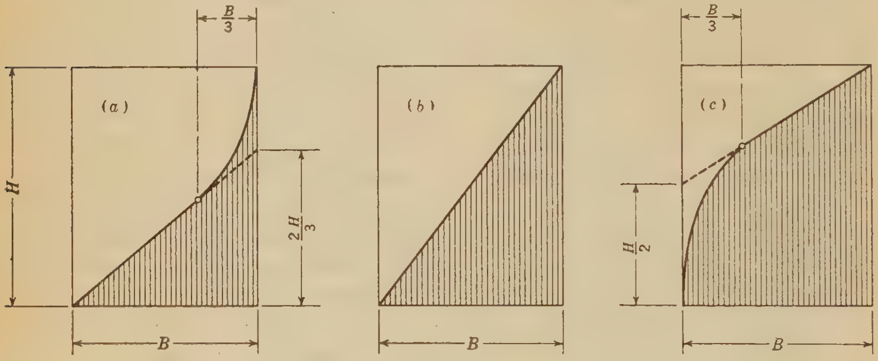

FIG. 22

Mr. Henny certainly deserves the gratitude of the profession for the clear presentation of an important subject and for the stimulating discussion which he has initiated.

D. C. HENNY,[65] M. AM. SOC. C. E. (by letter).—The voluminous discussion of this paper by a large number of men, foremost in the design and construction of dams, shows that the subject of dam stability is opportune. The writer feels under deep obligation to those who contributed to the discussion, which it is hoped may clarify new ideas and aid in arriving at reasonably correct conclusions.

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[65] Cons. Engr., Portland, Ore.

The discussion has covered so wide a field that in order to permit adequate comment it is expedient to deal with it under various headings.

Uplift Pressure.—The assumption that, for design purposes, uplift pressure may be safely estimated to diminish from full reservoir head to full tail-water head in a straight-line relation appears to have found general favor. Mr. Harza, however, argues that because of greater vertical loading, with reservoir full, toward the down-stream toe, the passage of water might meet with greater resistance and the pressure variation might thus exceed the straight-line relation between the up-stream and the down-stream pressure. A similar argument was advanced by Professor Terzaghi for the case of soft rock.

The argument itself is sound. Field measurements relating to reasonably sound foundation, however, fail to support it and show the pressure to be far less than that indicated by the straight line.[66] This is true whether the base was drained or whether drainage had been omitted.

Uplift Area.—In the matter of the relative area on which the uplift pressure acts, there is far less unanimity of opinion. The writer's arguments were: (*a*) That, with heel loading (reservoir full) greater than H, the area may be safely estimated at 25%; (*b*) that for a heel loading of 0.4 H this estimate should be increased to 40%; and (*c*) that for progressively lighter heel loadings the effective area might gradually reach 100 per cent. For design purposes he held that the heel loading should never be less than 0.4 H.

The lower limit of 25% was based on the reasoning that only pore uplift area of 12% need be considered in homogeneous material; but in view of the existence of joints between the rock and the concrete and of joints in the concrete itself, and also because of lack of homogeneity of both rock and concrete, the mentioned 12% should be increased to 25 per cent.

Mr. Creager states that the void area in any section may greatly exceed the void volume, but the reason therefor is not given. Mr. Hanna expresses the same opinion and presents a formula based on the assumption that pores may be represented by spheres and that a plane may be arranged such as will cut all spheres along a grand circle. He thus finds that, for 12% void volume, the void area in the plane is 30 per cent.

As regards the latter estimate it is submitted that if a plane with such a percentage of void area can be found, it would necessarily be very irregular and would, in various directions, cut through more intermediate solid concrete, or rock, than would be the case with a flat plane, thereby neutralizing the effect of passing through increased pore area.

In the case of a flat plane and homogeneous material the percentage of void area cut through must be the same as that of void volume. This can be shown readily by the following reasoning: Consider a cubical vessel to be completely filled with either rock or concrete. Assume the voids in this material to be evenly distributed and completely filled with water. Imagine the solid material to become fluid permitting it to settle and to allow the void water to rise to the top. Then, with a 12% pore volume, the heavier fluid will fill 88% of the depth and the water, 12 per cent. Now, consider any plane or line cutting through the mass from bottom to top; it is obvious that the area or distance passing through the water will be 12% of the total.

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[66] "Uplift Pressure in Masonry Dams: Measurements at Existing Structures," by Ivan E. Houk, M. Am. Soc. C. E., *Civil Engineering*, September, 1932, p. 578.

Mr. Woodard argues that in construction joints the cross-section of pores may be "many times" that in the adjacent concrete. Professor Terzaghi states that experiments made by him tend to show that porosity of concrete along rough shear planes may be as high as 95 per cent. The details of these interesting experiments were not stated, which makes definite discussion at this time impossible. If the inference is to be drawn that vertical forces exist in ordinary concrete dams of 95% of the base area times the uplift pressure it is difficult to explain the continued stability of many dams with dry slopes approximating and even falling short of seven to ten. At least to many engineers the very high tensile strength stated by Professor Terzaghi to be possessed by interporal columns of very small area can scarcely be a satisfying explanation, especially when applied to the plane of contact between rock and concrete. The unsucessful efforts in tests at the laboratory of the Bull Run Dam in 1927 and in like tests by the late Mr. Skafte, in Pasadena, Calif., to force water in a concrete construction joint (both cited in the paper) tend to contradict those findings, as does also the fact that in most of the uplift points placed in concrete construction joints in dams, no pressure, and frequently no seepage water, could be recorded. Since it was further stated that the said 95% holds good for 1:3 cement mortar and probably for neat cement, the Woodard experiments also fail to fall in line with such inference.

On the other hand, whether the assumption of 25% pore pressure area at the junction of rock and concrete is conservative, must remain a matter of opinion for the present. However, this issue loses some of its importance in case the practice is followed of avoiding horizontal areas of rock-concrete contact along which horizontal movement might take place and of preparing the rock surface as far as practicable in such a way as to present surfaces alternately normal and parallel to the direction of principal forces. This is for the reason that, while the result of all uplift forces is not thereby altered, a roughly horizontal contact plane along which uplift forces are concentrated and along which sliding might take place, is lacking.

The foregoing, reasoned on the basis of straight-line distribution, refers to the condition that the heel loading (reservoir full) is at least equal to H. When this loading is less and reaches a value of 0.4 H, the writer's assumption was that the effective uplift area may then be estimated safely at 40 per cent. The basis for this estimate is that numerous small dams have been built with a vertical water face and a down-stream slope of 0.70 and that such dams in spite of a heel loading of 0.36 H have never shown any progressive increase of percolation or uplift pressure. This argument is deemed by Mr. Creager to be inadequate. The writer might have added that many larger dams are in the same category, and that they have generally given no sign of increase of uplift or seepage. One exception is the St. Francis Dam, to which frequent reference has been made in the discussion but which, strictly speaking, is outside the conditions originally set, namely, that the dam foundation consists of solid rock not affected by water. Mr. Creager argues that all these narrow base dams may owe their safety to adhesion to the rock. This must be so, in fact, in case the 40% effective uplift area actually prevailed. For a loading of 0.4 H and 40% maximum effective uplift area, however, there would be no tension in the solid concrete, and adhesion need not be depended upon for safety. For the more probable smaller uplift area under conditions of heavier heel loading, a substantial balance of compression in the concrete must remain.

Mr. Jorgensen maintains that the 40% area estimate is too low in view of the uncertainties as to actual vertical pressure at the heel and also considering earthquakes. He also brought up the question of distribution of stresses which will be dealt with by itself. Mr. Jorgensen is correct in calling attention to the necessity, according to locality, of making special allowance for earthquakes in every part of the design. In stating that 40% was intended as the maximum uplift area estimated by the writer, Mr. Jorgensen would be correct only if he had added that the proportions of the dam are such that the minimum heel loading is 0.4 H. For smaller heel loading, conditions approaching those illustrated in Fig 5(b) must be anticipated. He mentions the 100% uplift area estimate proposed by Francis, which is equivalent to the Levy requirement and a dry slope of 0.843 (see Fig. 8), but the writer is aware of only a few high gravity dams in the United States built with such slope (Wachusetts and Pine Canyon Dams).

Referring to Table 6, Mr. Harza asks whether the percentages of area therein given, apply to solid concrete only or to foundation and construction joints as well. The pore area was raised from 12 to 25% because of lack of homogeneity of the concrete, such as is exemplified by joints at the base and at higher elevations. Finally, to cover uncertainties a further increase of uplift pressure area was proposed, as may be noted by comparing Columns (4) and (5) of Table 6.

Mr. Harza is correct in holding that no laboratory experiment can truly simulate imperfections of joints in the field and that no formula or table can dispense with judgment gained by experience.

Uplift Force.—On the whole, no definite suggestion has been made as to any desirable alteration in Table 6.

Mr. Hanna reasons correctly from his Equation (18) that the effect of uplift on the total shearing strength is relatively small, and estimates it at 8 per cent. The same general conclusion may be drawn from Table 8 by comparing the values in Item No. 9 (which indicate the increase in the shear factor of safety by the elimination of uplift) with the corresponding values in Items Nos. 10 to 15, which give the factors of safety including uplift. The effect of uplift on safety is thus shown to be from 3 to 7% for dams with a back slope of 0.707 and from 2 to 4% with a slope of 0.843, assuming reasonably tight and hard foundation. It is only with steeper back slopes when sufficient vertical loading does not exist at the heel, or where foundation rock is soft, seamy, or otherwise open to the easy penetration of water, that uplift may acquire dangerous proportions and begin to affect the safety of a dam seriously.

Classification of Foundation Rock.—Mr. Parsons and Professor Terzaghi propose the classification of foundation rock, the former as regards the percentages of effective uplift areas and the latter as to uplift pressure. Mr. Jorgensen advocates the fixing of a lower and upper limit of base width for gravity dams with reference to the effect of uplift for the guidance of regulatory bodies. German specifications, as mentioned by Mr. Knapp, prescribe 20, 30, or 40% of uplift area according to the quality of the rock, seemingly independent of loading.

The question of foundation in its relation to soundness, seaminess, and stratification appears to be a matter of study to be aided in each important case by a geologist, as stated by Mr. Knapp. Such co-operation has become common practice. General principles can be laid down and discussed, but for their proper application, good judgment and experience will always remain necessary.

The Coulomb Theory.—This theory, and its applicability to the determination of shear resistance, has been the subject of much discussion. The writer assumed its practical correctness under conditions of pressure loading not exceeding 1 000 lb per sq in. Mr. Floris states correctly that engineers use working stresses and not conditions of stress near the breaking point. Mr. Baumann refers to this theory in connection with what he terms the "crushing" test, having reference presumably to the ordinary cylinder compression test. He is right in considering this test intricate in view of friction at the cylinder ends with the machine plates, and of the development of radial tension under certain conditions. However, even if friction has the effect of end "hooping" and even if at times explosive breakage occurs along planes parallel to the axis, this need not necessarily interfere with drawing conclusions as to shear value from tests when breakage is clearly due to shear.

In Fig. 10, Mr. Baumann shows a barrel-shaped side bulging of test specimens. It may be of interest to give the results of experiments in the laboratory of the U. S. Reclamation Bureau at Denver, Colo. Poisson's ratio was determined for 48 cylinders, 2 by 4 ft, and 28 cylinders, 3 by 6 ft, of various mixes, at the quarter-points as well as in the middle. The heads of the cylinders were not greased. The average values for loadings of about 1 000 lb per sq in. were, as follows:

Location	2 by 4 ft	3 by 6 ft
Upper quarter-point	0.216	0.211
Middle	0.183	0.181
Lower quarter-point	0.187	0.192

It was found that not only is the middle diameter the smallest in the summary of results, but the same is the case for practically every individual test. The point that the writer wishes to make is that such bulging as is shown in Fig. 10, is not determinative of Poisson's ratio, but occurred near the point of failure (as, in fact, was stated by Mr. Baumann) and resulted, in all probability, from increased plastic flow which always precedes failure.

Mr. Merriman holds that every failure of a brittle material involves tension and considers the writer's combination of the Coulomb internal friction theory with the basic formula, to be simply the establishment of a ratio of convenience between shearing strength and water loading. Mr. Knapp, on the other hand, holds that the Coulomb theory is applicable to dam design within the range of shear stresses usually found.

In the remarkably comprehensive "Study of the Failure of Concrete Under Combined Compressive Stresses," to which previous reference was made by the writer, the failure of concrete under compression is summarized,[67] by one of the authors, A. Brandtzaeg, Assoc. M. Am. Soc. C. E., as follows:

> "Some essential features of the breakdown of concrete in compression suggest themselves. It appears that the departure from elastic action begins as a plastic sliding failure along small inclined surfaces scattered throughout the material. It seems reasonable that the resistance to bond slip or plastic sliding may be partly of the nature of adhesion and partly an internal friction, * * *."

The subject of the paper was not a study of the details of concrete failure, however, but an approach to evaluating shear resistance before failure takes place, and until a better theory is developed and demonstrated it is believed that the Coulomb theory is the one most nearly fitting the requirements in the study of shear stability of dams.

Dr. N. Kelen has made shear experiments,[68] which were referred to briefly by Professor Terzaghi and Mr. Floris. He cast early-strength, cement concrete blocks on rock blocks with the contact surface roughened, and, after seven days, sheared them apart under various loading conditions, including zero load. The apparatus (which is remarkably similar to that suggested by Mr. Knapp in Fig. 14) is such as to bring the outside shearing forces exactly in the plane of shear (see Fig. 23). The contact area between blocks was

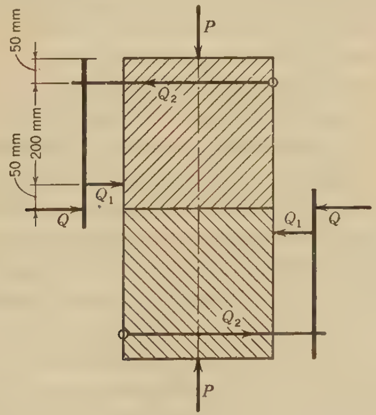

FIG. 23

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[67] *Bulletin No. 185*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill., pp. 87–88.

[68] "Versuche zur Bestimmung des tangentialen Sohlenwiderstandes von Gewichtsstaumauern," 1933; received February 4, 1934, by courtesy of Mr. Nelidov.

about 70 sq in.; the average vertical loading, centrally applied, ranged from 0 to 400 lb per sq in.; and, with the load applied at the edge of the middle-third, the maximum loading ranged from 0 to 800 lb per sq in.

Dr. Kelen's Table No. 3,[68] leads to the formula for shear, $s = 275 + 0.86 \times 2q$, which, in form, is identical with the writer's Equation (3). He found a value of k for eccentric loading, $2 \times 0.86 = 1.73$. In three experiments with central loading the average value found by him is 1.40. These values seem relatively high, but it should be remembered that they apply solely to shearing resistance along contact planes between concrete made with special early strength cement and rock, with its surface specially roughened.

Professor Terzaghi reports that the value of k developed by Kelen, decreases materially for higher loads. The actual values are, as follows:

Unit loading, in pounds per square inch	k, eccentric loading
80	1.80
159	1.76
313	1.58
392	1.77

The writer would argue from these figures that there is practically no decrease in the value of k within the range of these experiments.

Professor Terzaghi also quotes the formula for shear, $s = 430 + 1.13\ q$, found by Messrs Hertwig, Ludin, and Petermann for 1 to 5 concrete in torsion tests made in 1933.

The Kelen and Hertwig experiments should go far in disposing of the question raised by Mr. Nelidov as to whether shear and so-called sliding act simultaneously. The fundamental reasoning in the writer's mind has been that molecules are forced into closer proximity by compression and that thereby shear movement is impeded. The reasoning of Professor Brandtzaeg covers the subsequent stage when plastic sliding has begun.

Mr. Davis refers to Dr. Stucky's investigation of the failure of the Gleno (multiple-arch) Dam, in 1932, due to buttress shear in connection with a poor quality of concrete. In his report, Dr. Stucky deducts the frictional component of shear (taken at 0.75 times the compressive stress at right angles to the plane considered) from the total shear resistance and thus arrives at what he terms residual shear. This indicates a general acceptance by him of the Coulomb theory.

Reference was made by Mr. Nelidov to shear tests made by Mr. Lewis H. Tuthill. Mr. Tuthill found the shearing strength of concrete joints (new concrete being placed on old at 3-day intervals and then broken at the end of 11 and 12 weeks) to increase from an average of 437 lb per sq in. under conditions of no loading to 1 102, 1 108, and 1 030 lb per sq in., with respective loadings of 104, 208, and 312 lb per sq in. The tests are in the nature of punching shear tests. Judging from the manner in which they were made, it seems certain that tension controlled failure. The most suspicious feature of these tests is the large increase of shearing strength from no load (437 lb per sq in.) to total load (1 102 lb per sq in.), with the unit load on the plane of shear equal to 104 lb per sq in., which would be an inordinately high value of $k = \frac{1\,102 - 437}{104} = 6.4$.

In commenting on the punching shear tests quoted by the writer,[4] Mr. Glover states that he does not consider the assumption of uniform shear distribution warranted. The writer referred to these tests merely to show that the average shearing strength, no matter how distributed, increased with the pressure on the plane of shear; he did not use their results for quantitative determination. The same argument is valid against the Woodard tests illustrated in Fig. 2 (*b*) and Fig. 2 (*c*), and the Tuthill tests. The results of these tests were not used by the writer for purposes other than to show trends.

Mr. Glover also argues that other tests quoted are inadequate for determining values of s_1. He refers in this connection to Table 1 and takes special exception to Line 2 in which a value for s_1 in the case of certain concrete is found to be 783 lb per sq in. on the basis of the Coulomb theory. He further assumes the tensile strength of the concrete tested to be 300 lb. per sq in. On the basis of these assumptions and, after finding that a shear value, under no load, of 783 lb per sq in., can be compatible only with a tension of 407 lb per sq in., he concludes that such tests are inadequate.

The writer calls attention to the fact that the example selected by Mr. Glover, with $\alpha = 62° \, 30'$, represents a border condition. Measurements of the angle of cone break in testing 136 cylinders, 6 by 12 in., of ages varying from 28 to 365 days, show that 117 (or 86%) broke at angles greater than $62° \, 30'$ and that the average angle of break was 67 degrees. For this condition (with $s_1 = 636$ lb per sq in. (see Table 1), and following the same reasoning), he would have found a corresponding tension of 270 lb per sq in., which is well below his assumption of 300 lb per sq in., tensile strength.

In his discussion, Mr. Woodard has contributed very valuable information regarding breakage tests with mortar prisms, additional to that mentioned by the writer, and has called the latter's attention to what he deems to be a mistake in Table 3. It refers to the test break illustrated in Fig. 2 (*b*). For this reason and also because great uncertainty must exist as to the distribution of shear and tension stresses in both cases, shown in Fig. 2 (*b*) and Fig. 2 (*c*), Mr. Woodard's comments must be regarded as casting some doubt on the validity of the last two lines of Table 3.

Shear Safety Factor.—The proposition advocated by the writer to take into consideration resistance due to shear when studying the danger of downstream movement of a gravity dam under the water load has met with rather full approval. Mr. Parsons agrees that "shear is a more important factor than sliding"; Mr. Eremin merely holds that "Coulomb's internal friction theory" is a convenient guide, alongside the middle-third and the sliding-factor theories; and, Mr. Davis agrees that the substitution of the factor of safety against shearing failure for the sliding factor is logical.

Mr. Creager also gives the shear method of judging safety his general approval. He deserves great credit for having called attention in 1931[69] to the fact that the factor of safety against down-stream movements is much less for high dams than for low ones, due to lesser shear resistance. Mr. Hanna gives conditional approval acknowledging the importance of shear resistance, while Mr. Jorgensen holds that shearing strength may add considerably to the factor of safety, especially where there is compression normal to the plane of shear.

Mr. Nelidov is in agreement, but advocates the use of great caution because of lack of test data; Mr. Baumann holds the shear safety factor to be of greater importance than the sliding factor, considering horizontal movement; Mr. Harza agrees with the writer's general argument for shear factor instead of sliding factor for dams on non-stratified foundations; while Mr. Godfrey and Mr. Knapp impliedly approve the argument. Mr. Jacobs holds that shear as a criterion should not take precedence over sliding. It is only Mr. Karpov who believes that the middle-third method, the sliding factor method, and the proposed shear factor method are all arbitrary, that none is proved, and that an attempt to prove them will give results contrary to expectation. In support of this belief, he argues that the writer made an attempt to introduce the "non-existent imaginary constant shear," which is obviously the same as the unloaded shear measured by Dr. Kelen and Mr. Hertwig, the residual shear mentioned by Dr. Stucky, and the term, s_1, in the writer's Equation (3). Quite properly, Mr. Nelidov cautions against the haphazard use of formulas. This is particularly true as regards the shear factor formula, Equation (6). The values of $s_1 = 400$ lb per sq in. and $k = 0.7$ in the numerical examples in Table 8, are suggestive only in the case of good concrete and hard rock, and the writer wishes especially to emphasize that the greatest danger is commonly at the foundation because shear stresses are highest there, and rock shear strength may be relatively low. The value of $s_1 = 400$ lb per sq in., is probably reasonably safe to use for good concrete and rock, including the effect of uneven distribution of shear; but for soft rock this value may be far too high. In connection with the design of an important dam, the writer recently recommended, as a result of rock shear experiments, the equation, $s_1 = 40 + 0.45\ q$.

The writer admitted quite freely that experimental data as regards shear are signally insufficient and that there is great need of extended research which he is pleased to note is likewise urged by practically all those who discussed this phase of the paper. He believes, however, that enough is known to justify the application of the shear factor to design, after shear resistance of rock has been determined under various loads, the latter being superfluous if compression tests of rock specimens clearly indicate the usual superiority of strength of rock over that of concrete.

Uneven Distribution of Shear.—The principal objection raised in the discussion to the use of the shear safety factor is that it contains the error involved in assuming a straight-line distribution of forces. On that point it was stated in the paper that "in ordinary cases the maximum shear stress will always be less than twice the average and this range is fully covered by such assumptions as were made in the study." This is equivalent to stating that the local factor of shear safety is usually more than 50% of the average. Some check on this statement is possible by the use of the experimental data recorded by J. S. Wilson and William Gore[70] who present curves showing the distribution of horizontal shearing stress and vertical pressure in and on the base of a rubber model of a dam 127 ft high. The dam differs little from the standard triangular section discussed in the paper, except for the introduction of large fillets at both toes, such as are mentioned by Mr. Karpov. Fillets have been used for years in Germany and many engineers, including the writer, have favored their introduction in the United States. The down-stream slope of the model is 0.695 ignoring fillets, and, therefore, is somewhat steeper than that necessary to make the vertical heel loading at least 0.4 H on the basis of straight-line distribution. The vertical loading as scaled is shown in Column (2) of Table 13, from which, on the basis of Equation (3) with $k = 0.7$, there would result for the second part of the equation, values as shown in Column (3). Assuming in Equation (3) a value of $s_1 = 400$ lb per sq in., the resisting shear strength is given in Column (5) while Column (6) shows the measured shearing stresses. The individual factors of safety at various points are indicated in Column (7) the lowest being 11.0, at the nominal down-stream toe, which is 65% of the average. This materially exceeds 50% and, therefore, confirms the foregoing statement.

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[69] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 197.

TABLE 13.—FROM MODEL TESTS BY WILSON AND GORE (PROTOTYPE $H = 127$ FEET; FILLETS AT BOTH TOES; BASE WITHOUT FILLETS, 88 FEET; $N = 0.692$; NO UPLIFT.)

Distance from up-stream vertical face, in feet *	EQUATION (3)			SHEAR		SHEAR FACTOR		
	Vertical stress, q, in tons per square foot	Product, $k\ q$, in tons per square foot	Value of s_1, in tons per square foot	Strength, in tons per square foot	Stress, in tons per square foot	Local	Average	Local *vs.* average (percentage)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
—20	0.4	0.3	28.8	29.1	0	∞	17.0	....
—15	0	0	28.8	28.8	2.4	12.0	17.0	70
—12	—0.4	—0.3	28.8	28.5	2.3	12.2	17.0	72
—10	0	0	28.8	28.8	2.1	13.7	17.0	80
0	2.2	1.5	28.8	30.3	1.8	16.8	17.0	99
22	4.5	3.1	28.8	31.9	0.9	35.4	17.0	204
44	5.5	3.9	28.8	32.7	1.8	18.1	17.0	113
66	5.2	3.6	28.8	32.4	2.8	11.6	17.0	68
88	2.6	1.8	28.8	30.6	2.8	11.0	17.0	65
110	0.7	0.5	28.8	29.3	1.8	16.3	17.0	96
121	0	0	28.8	28.8	0	....	....	....

* From —20 to 0 ft, up-stream fillet base; from 0 to 88 ft, main base; and, from 88 to 121 ft, down-stream fillet base.

Table 13 indicates that if foundation rock had qualities identical with the superimposed concrete as was the case in the model, there may be some tension under the up-stream toe fillet, the tension in this case being less than 6 lb per sq in. For a dam with a minimum of $n = 0.707$ instead of 0.692, this tension (if it continues to exist at all) is negligible, especially in case large fillets are provided. As far as safeguarding against uplift by satisfactory up-stream toe pressure is concerned, it should be noted from Table 13 that, at the point in the vertical from the top, 11 ft from the point of tension, a compression of 2.2 tons per sq ft, was found equivalent to about 0.56 H, 40% above the minimum limit suggested by the writer.

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[70] "Stresses in Dams; An Experimental Investigation by Means of India-Rubber Models," by J. S. Wilson and William Gore, *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Pt. II (1907–08), p. 107.

Mr. Knapp claims that a scientifically more accurate shear safety factor would be the relation of local shear strength to local shear stress at whatever point in the dam this relation is found to be a minimum, which would probably be near the down-stream toe. The same claim is found to have been made by Dr. Kelen.[68] The writer questions the validity of this claim as regards the up-stream toe. The distribution of shear as found in the model test of Wilson and Gore is typical of what is found by the use of elasticity equations on actual dam design, in that there usually is a sharp rise in relative shear resistance in a down-stream direction in that locality which would quickly put a stop to the spreading of any initial horizontal crack.

The case of the down-stream toe is different. The heavy shear stress in that locality, caused by high horizontal stresses against vertical faces, will produce relatively high shear deformation, and thus will automatically reduce shear stress; but if, in spite of such reduction, a shear break occurs, progressive shear breaking may take place.

In the model test illustrated by Table 13 the lowest local shear safety factor at the down-stream toe (Station 88), as stated previously herein, was found to be 65% of the average. It can scarcely ever be less than 50% of the average; hence, an average factor of 2 should prevent all initial shear failure mentioned by Mr. Karpov. The writer used 4 as a minimum in conjunction with a low estimate for s_1, which should guarantee against the beginning of a shear crack.

The reasons for having suggested the use of average rather than local shear safety factors are: First, that the determination of the latter is laborious and rather uncertain in its results because of lack of definite knowledge of the elastic properties of mass rock; second, if the average factor is held at not less than 3 (preferably at 4) and if in conjunction therewith the value of unloaded shear strength, s_1, in Equation (3) is estimated conservatively (as, for instance, 400 lb per sq in. for sound rock and good concrete) no danger of progressive shear failure can exist; and, third, that the average factor is believed to give a broader view of safety than the local factor, if it is applied with understanding.

The general argument advanced by the writer in favor of considering shear in ascertaining safety is in any case not vitiated by the choice of detailed method and the designer can obviously suit himself as to the use of a local shear factor or an average shear factor. The necessity for determining local shear factors, even though laborious and uncertain in its results, was not only admitted, but urged, by the writer in the case of very high dams.

Mr. Knapp found, in the instance quoted by him, shear safety factors about one-half those estimated by the writer in Table 8. This is only natural because, in his determination, he used twice the average shear stress in conjunction with the writer's estimated shear strength, which had been greatly reduced from measured values because it was to be used in connection with the straight average shear stress.

Sliding Factor.—The sliding factor as a measure of resistance to down-stream movement continues to have strong advocates. Mr. Houk, for instance, holds that sliding factors should be computed in the future as in the past. It may be pointed out that Equation (3) covers the cases where the sliding factor becomes of real importance; that is, when s_1 becomes small or negligible and only the term, $k\ q$, remains. This last term then becomes expressive of simple frictional resistance, and k becomes the ordinary sliding factor.

The instinctive reasoning of an engineer is that in the sliding frictional resistance he has found an irreducible minimum, which should permit the use of low factors of safety. The reasoning would be more valid if coefficients of friction like the weight of masonry and water could be known definitely. One instance comes to the writer's mind of a concrete gravity dam, the original design showing a sliding factor of 0.55. This was deemed safe, until it was shown that the horizontally stratified shale foundation allowed percolation along contact planes, resulting in a tested friction coefficient of 0.30.

There is about as much uncertainty concerning coefficients of friction as there is about shearing strength. Therefore, the use of reasonable factors of safety cannot be dispensed with in such cases as dams built on gravel and sand, on soft rock, on horizontally stratified foundation, or on poorly bonded horizontal construction joints in the concrete or masonry.

Concrete and Foundation Joints.—Contact surfaces between rock and concrete and between concrete of different ages are sources of weakness. Mr. Creager states that the surface of foundation rock should be roughened or keyed, and he likewise advises the use of keys in concrete construction joints. Mr. Jorgensen advises stepping, and others, sloping upward in a down-stream direction.

The writer believes that since down-stream movement must be largely horizontal, horizontal surfaces should be avoided as much as practicable. It has been pointed out in the discussion that because of the different nature of rock and concrete and thermal movement along contact planes, adhesion is uncertain. The low value of unloaded shear found by Dr. Kelen (275 lb per sq in.) may be an indication of this. Therefore, the foundation might best be prepared saw-tooth fashion with surfaces normal and parallel to the directions of principal stress—in other words, along planes of no shear. If these saw-teeth acquire large dimensions, grout pipes should be placed so as to reach the deepest points of the excavation since, in the process of concrete cooling, there may be a tendency of weight transfer from low to high rock points and a pulling away of the concrete from the former. If this is done, the relative weakness of rock to concrete bond is of lesser importance, because shear, if it occurs, must pass through all rock, all concrete, or a combination of both. Similar joints have been used recently as vertical contraction joints in the concrete where these joints run parallel to the water face of the dam.

As regards concrete construction joints between layers, laboratory tests indicate that with good work they may be from 10 to 30% weaker than monolithic concrete. The smaller value was found as a result of compression tests in the Denver Laboratory, with compound, 6 by 12-in. cylinders. The lower half of the specimens consisted of a previously broken cylinder selected for showing a clear cone-shaped shear break. The upper half was cast on the lower half; it consisted of fresh concrete, which was left to set 28 days, the rebuilt cylinder being then subjected to the standard compression test. The average breaking loads, in pounds per square inch, at 28 days, were as follows:

Original cylinders......................	3 450
Compound cylinders....................	3 030
Control cylinders.......................	3 280

As compared with the original cylinders the compound ones broke at 10% smaller load. In comparison with the control cylinders, which were cast entirely with the new concrete, the loss of strength was only 5 per cent.

The higher estimate of 30% loss of strength results from tests made by R. E. Davis, M. Am. Soc. C. E., at Berkeley, Calif., comparing the modulus of rupture in monolithic concrete with that in beams having a joint between old and new concrete. In this way Professor Davis found, in an indirect manner, a reduction of 30% rupture strength, and (by inference) a similar reduction of joint strength as compared with monolithic concrete.

Adhesion tests were made with 14-in. cylinders in the laboratory at the Bull Run Dam, 31 days after casting new on old concrete of a 1:4.2 mix (aggregate, 0 to ¾ in.; and water-cement ratio, 1.22 by volume). The average monolithic tensile strength was 130 lb per sq in., and three compound cylinders pulled apart, respectively, at 105, 102, and 95 lb per sq in. (average, 101 lb), being an average reduction of 23% in tensile strength.

These percentages cannot be fully expressive of conditions in field joints dependent as they are upon the character of the field work, and at times the reduction of strength may easily be much greater. The small percentage of adhesion area between rock and concrete, as found by Joseph Wright, M. Am. Soc. C. E. (quoted by Mr. Creager), must be due to very exceptional causes.

The danger of joints inclined down stream was properly pointed out by Mr. Baumann in respect to the co-operation of gravity and uplift in tending to produce shear. Such joints can readily be avoided in concrete and where the rock is found sloping downward in a down-stream direction there is every reason for stepping it with reference to the direction of principal stresses.

Overturning.—On this subject the discussion has included various comments, as follows: Mr. Creager remarks that a usual factor of safety against overturning must rely on tension; Mr. Jorgensen deems a dam, with a base to height relation of ⅔—assuming no uplift, no earthquake, and linear stress distribution—to have an overturning safety factor of 1; and Mr. Nelidov deems an overturning factor of safety of 1.5 satisfactory in case bond is disregarded. The meaning of factor of safety in this connection seems somewhat vague.

For the ideal triangular section of dam the moments about the down-stream toe are for water pressure, $P\,p = \frac{1}{6}\,H^3$, and for $W\,w = 0.8\,H^3\,n^2$. The overturning safety factor, Q_0, with uplift force within the limits of vertical loading, will thus be $\frac{0.8\,H^3\,n^2}{\frac{1}{6}\,H^3} = 4.8\,n^2$. Taking Mr. Jorgensen's case, for which n equals 0.66, Q_0 would be 2.1 without uplift and not 1 as stated by him. In such a case, however, uplift cannot be ignored and the overturning factor may easily become less than 1.

Little prominence was given to this question in the paper because it seemed obvious that danger from overturning is automatically guarded against by compliance with other requirements. For instance, a minimum heel loading of $0.4\,H$, as suggested in the paper, in the case of good rock foundation, gives a minimum value of n of 0.707; and, hence, $Q_0 = 4.8 \times 0.707^2 = 2.4$. In the case of badly seamed rock, safety against shear demands a base length as indicated by the solid line curve in Fig. 5(*b*). For a 280-ft dam, for instance, n equals 0.80 and Q_0 becomes 3.1.

High values of Q_0 like 4 are believed to be unnecessary because the only elements involved are weight of masonry (which cannot suffer appreciably from deterioration) and weight of water, both being definitely known. Values of Q_0 are merely indexes relating to design and have otherwise no meaning since long before overturning can occur (in all but very exceptional cases) other dangers arise, such as crushing of concrete on foundation material at the down-stream toe, uplift increase, destruction of initial shear resistance, and flotation.

As far as the writer knows the only instance of actual overturning of a dam was furnished by the failure of the Bouzey Dam, mentioned by Mr. Jorgensen. The ordinary flotation failure was converted in this case into overturning by the presence of a deeply founded, heavy, concrete block which resisted sliding and, under the effect of uplift force induced by a leaky foundation, acted as a down-stream toe hinge. Otherwise, all notable failures appear to have been due to sliding on foundation layers or on the base after shear had been destroyed, aided by flotation due to uplift.

Research.—It has been contended that present-day knowledge of shear in concrete and rock is not sufficiently advanced to warrant placing dependence on any stability formula in which shear is a factor. It might be argued with even greater force that too little is known about the character of a foundation and the rock composing it to warrant studies of elastic behavior in the zone of contact. The writer would conclude that admitting the limits of present knowledge designers should use such tools as they have as long as results seem logical and are confirmed broadly by experience. Their imperfection should not lead to their abandonment, but to the use of caution and to extended research.

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ESTIMATING THE ECONOMIC VALUE OF PROPOSED HIGHWAY EXPENDITURES

BY THOMAS R. AGG,[1] M. Am. Soc. C. E.

WITH DISCUSSION BY MESSRS. W. W. CROSBY, ROGER L. MORRISON, SAMUEL B. FOLK, W. S. DOWNS, GEORGE E. MARTIN, J. T. L. MCNEW, H. E. PHELPS, C. C. WILEY, AND THOMAS R. AGG.

SYNOPSIS

This paper presents a discussion of a method whereby a proposed highway improvement can be analyzed to determine whether the required expenditure is economically justifiable. Three types of highway improvement projects are analyzed and a typical example of each case is presented. These three types are:

1.—Improvement of a highway by the construction of a roadway surface of a higher type, including the necessary supplementary work.

2.—Improvements by means of relocations that shorten the distance between terminals.

3.—Improvements that reduce the rate of grade, with or without a change in distance.

While it is recognized that certain types of highway improvement present themselves, in which the dominating purpose is the removal of the accident hazard, the major portion of the construction work of State Highway Departments and of cities consists of projects that should go forward only when they are justifiable on economic grounds. Strictly speaking, the removal of the accident hazard is economically justifiable, but the analysis of such cases presents many difficulties, and will not be dealt with in this paper.

NOTE.—Presented at the meeting of the Highway Division, Atlantic City, N. J., October 10, 1932.

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Basis of the Analysis

The criterion by which to determine whether a proposed highway improvement is economically justifiable is one of annual costs. In certain cases highway improvement may result in a lower annual cost of maintenance, using the term, "maintenance," in the broad sense of including all the cost incident to the perpetual maintenance of the highway in a standard serviceable condition. The greater part of major improvements, however, do not result in a reduction of the annual cost of highway maintenance and must be considered on broader grounds, in order to justify the necessary expenditures.

The principle that the ultimate purpose of highway improvement is to permit highway transportation at the minimum cost must be introduced into the analysis in most cases; that is, the improvement should result in a lower annual cost of transportation if it is to be justified on economic grounds. The cost of transportation consists of two elements—the road cost and the vehicle cost—and while certain types of improvements may actually increase the annual road cost, they decrease the annual vehicle operating costs sufficiently to more than offset the increased road costs. The net result is transportation for the traffic known to use the highway, at a cost less than that incurred before the expenditure for the highway improvement.

In a measure, the production idea may be introduced into road building. In manufacturing, the efforts of management are directed toward the production of a particular commodity at the minimum cost. Improved machinery or changed methods of production are introduced only after an exhaustive study to determine their effect on the cost of production, and if that cost is not decreased by the proposed changes sufficiently to justify the necessary expenditure, the old methods are continued in use. In highway building, a somewhat similar criterion may be set up. The actual cost of transportation on an existing highway may be ascertained with reasonable accuracy. The probable cost of transportation after a proposed improvement has been completed may be estimated within reasonable limits. If the correct method of analysis is followed and if values are obtainable for all the required factors, the results indicate quite conclusively whether or not the expenditure should be made.

The fundamental theory involved in the computations that will be used in any analysis of the character under discussion will be set forth briefly. Those competent to judge in matters of this nature are in substantial agreement with the material that is presented herein.

The starting point in the development of the necessary theoretical basis for economic comparisons in highway work is the simple statement that:

$$\text{Highway transportation costs} = [\text{Highway costs}] + [\text{Vehicle costs}]$$

The application of the principle thus set forth to a specific case requires that each of the groups of costs be ascertained and that they be reduced to some common unit so that they can be added to give a correct unit cost of transportation. Probably the most convenient unit would be the ton-mile, which is used in railway practice, but the nature of highway traffic does not readily lend itself to the use of ton-mile costs. The use of the vehicle-mile has developed gradually as a measure of highway traffic, and although such a unit is open to criticism from several standpoints, it will be used in this paper simply because it is already well established and generally understood. The general definition for the cost relationships needed for economic studies of highway transportation may be expanded somewhat, as follows:

$$\begin{array}{c}\text{Comparative}\\ \text{cost of highway}\\ \text{transportation}\\ \text{per vehicle-mile}\\ \text{on a specified}\\ \text{highway}\end{array} = \left[\begin{array}{c}\text{Highway}\\ \text{costs per}\\ \text{vehicle-mile}\\ \text{of traffic}\end{array}\right] + \left[\begin{array}{c}\text{Cost of}\\ \text{vehicle op-}\\ \text{eration per}\\ \text{mile of}\\ \text{travel}\end{array}\right] - \left[\begin{array}{c}\text{Contribu-}\\ \text{tion by}\\ \text{vehicle to}\\ \text{road funds}\end{array}\right] . \quad (1)$$

The highway costs that must be considered are all the costs of all kinds incident to the construction, maintenance, and administration of the highway. This may be expressed, as follows:

$$\begin{array}{c}\text{Annual}\\ \text{highway}\\ \text{costs}\end{array} = \left[\begin{array}{c}\text{Interest on}\\ \text{the invest-}\\ \text{ment}\end{array}\right] + \left[\begin{array}{c}\text{Annual cost}\\ \text{of routine}\\ \text{mainte-}\\ \text{nance}\end{array}\right] + \left[\begin{array}{c}\text{Annual ad-}\\ \text{ministration}\\ \text{costs}\end{array}\right]$$

$$+ \left[\begin{array}{c}\text{Annuity for}\\ \text{depreciation}\end{array}\right] + \left[\begin{array}{c}\text{Annuity for}\\ \text{periodic re-}\\ \text{pairs at in-}\\ \text{tervals of}\\ n' \text{ years}\end{array}\right] + \left[\begin{array}{c}\text{Annuity for}\\ \text{periodic re-}\\ \text{pairs at in-}\\ \text{tervals of}\\ n'' \text{ years}\end{array}\right] + \ldots\ldots (2)$$

It should be obvious that Equation (2) may be converted to any unit desired; but for most purposes it will be found convenient to reduce these costs to annual highway cost per mile per unit of traffic.

In any discussion of the annual cost of highways the question arises as to the application of interest to the computations. The position taken in this presentation is that proper account of interest must be introduced if strictly correct conclusions are to be drawn. The facts that no interest is actually paid out on funds used currently, and that interest is paid out when improvements are financed by bond issues, seem to be confusing to those who have studied this subject. As a matter of fact, it is of no significance in this connection whether the improvement is financed by current funds or by a bond issue. If the public finances a project from current funds, it has invested a certain sum in a utility from which it expects to obtain a service in lieu of a direct financial return. Had that sum remained in the hands of the individuals who comprise the public, each might have invested his share in a dividend or interest paying security. The highway, therefore, should be charged with interest on that part of the investment that represents deferred service and at the rate of interest the funds could undoubtedly earn if gainfully invested. That is exactly the rate the public must pay for money borrowed, since Government bonds constitute the safest investment available to the public.

When the financing is through a bond issue, the interest charged in computing the cost of highway service is actually paid out, but the public does not make its contribution to the cost of the road until a bond is paid and then interest ceases. Meanwhile, each individual may gainfully invest the money he will contribute later to the road-bond fund and if he is thrifty he can cause the money to earn as much interest as he will later pay; for example, he may purchase a road bond.

For these reasons the interest factor is introduced in the formulas set up herein, and in those applying to road costs the rate of interest is that borne by road bonds, which is taken at 4%, but may be some other rate in specific cases.

The relationships set up in Equation (2) may be written in the form of an algebraic equation and, for purposes of illustration and to simplify the presentation, an equation applicable to the road surface only is, as follows:

$$C = Ir + \frac{(I-S)\,r}{(1+r)^n - 1} + M + A + \frac{E'\,r}{(1+r)^{n'} - 1} + \frac{E''\,r}{(1+r)^{n''} - 1} + \ldots \quad (3)$$

in which,

C = the annual cost of any section of highway surfacing;
I = the initial cost of the surfacing, including engineering and administrative overhead;
r = the rate of interest adopted for the analysis (expressed as a decimal);
S = the residual or salvage value of the road surface when it is superseded by another type;
M = the annual maintenance cost for routine work that is paid for annually;
A = the annual administrative costs chargeable to the highway in question;
E' = the cost of periodical maintenance that is performed and paid for at intervals of n' years; and
E'' = the cost of periodical maintenance that is performed and paid for at intervals of n'' years. (This factor applies to improvements of a different character from those designated by E').

In all ordinary cases the factor, A, may be omitted from Equation (3) without significant error. It would be included in special cases, such as the Holland Tunnel and certain toll bridges.

A little consideration of Equation (3) will disclose that it provides a means of estimating the annual cost of the maintenance of a road surface in standard condition for a long period in the future as well as of determining the historical cost of a specific road surface.

An equation of like form can be used for estimating the annual cost of each of the additional elements of a highway, such as drainage structures, retaining walls, elevated structures, tunnels, earthwork, guard fences, and warning and direction signs. However, it will be found that some of these elements have such great life expectancies that it will introduce no appreciable error if they are considered to have unlimited life under appropriate maintenance. Other elements, such as guard fences, may be considered to have unlimited life under proper maintenance even if each unit may be completely replaced piece by piece over a comparatively few years.

Having determined the annual cost, C, of any section of highway, including all its elements, that cost, by obvious methods, can be reduced to C_a, the annual cost per mile per unit of traffic. This is comparable to the toll that would have to be exacted if the highways were private enterprises and were operated to give the investors a return at the rate of 4% per annum.

The cost of vehicle operation may be derived by a process analogous to that used in estimates of highway costs, and need not be discussed in detail herein.

The foregoing discussion has been limited, for simplicity, to a single element of the road surface and a single vehicle. For economic comparisons it is necessary to take into account the costs of all elements of the highway under consideration and of all those that accrue to the traffic, and to forecast the traffic and expenditures on the highway during a reasonable period in the future.

Future highway costs can be estimated with considerable accuracy, but the estimation of the various elements of vehicle-operating costs presents a most illusive and troublesome problem. Vehicle design, tire durability, fuel and oil costs, insurance rates, wages, mileage of operation, and speed of operation, are all factors in the problem. The cost of each of these elements is exceedingly variable in any community, and it might seem hopeless to make any analysis that would be of significance. However, by making careful comparative measurements with a few vehicles it has been possible to set up relative cost figures that certainly apply to test vehicles. It is probable that the same general relationships apply to all traffic although the unit costs vary widely. Studies of this subject have been under way at several places for a number of years and by piecing together the bits of information gleaned from these various researches certain relative figures on the cost of vehicle operation have been set up.

In order to discuss the relative costs of transportation on various kinds of road surfaces it seems expedient to use the grouping now so widely adopted by highway engineers, namely, high type, intermediate type, and low type. While this is convenient for a general discussion, it is important to recognize that road surfaces cannot be classified by name only, since there is great diversity in the physical characteristics of any one of the road surfaces as it is constructed in various parts of the United States. Therefore, the types must be grouped on the basis of their relative physical characteristics.

On the basis of the foregoing classification certain estimates of the relative cost of vehicle operation were developed and published in 1928, as an outgrowth of a series of investigations of the various factors involved. The significant results[2] are given in Table 1.

From a recent examination of data now being developed by the writer, he has reached the conclusion that, for economic studies, it is on the side of conservatism to assume that the change from one type to the next higher, as these are defined in this paper, saves to the automobile about one-eighth of the operating cost on the lower type. This amounts to 0.8 cent per mile of travel for the "average" automobile on the basis of 1931 operating costs.

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[2] "Operating Cost Statistics of Automobiles and Trucks", by T. R. Agg and H. S. Carter, Members, Am. Soc. C. E. *Bulletin 91*, Iowa Eng. Experiment Station, Ames, Iowa, 1928.

For other classes of vehicles, particularly for trucks, it is necessary to deal with the actual composition of the traffic on a highway rather than with an average operating cost figure, since the character of the equipment in use in different parts of the United States is exceedingly diverse, and the composition of the commercial vehicle traffic reflects many local factors. The data at hand are too fragmentary to permit drawing any very positive conclusions as to the saving to commercial vehicles through change in type of road surface.

It is believed that no serious error will result if economic comparisons are based on a value of 17 cents per vehicle-mile of travel as the average operation cost of commercial vehicles on paved roads, and a saving in cost, by a change from an intermediate type to a high type, of 5 cents per vehicle-mile of travel. This saving results from lowered operating costs and an allowance for the value of the increased average speed that is possible. It is also believed that commercial vehicles with a capacity of less than 1 ton should be counted as automobiles, and that the commercial vehicle traffic should be considered as made up of commercial vehicles having capacities of 1 ton, or more.

TABLE 1.—RELATIVE COST OF OPERATING AN IMAGINARY "AVERAGE" AUTOMOBILE ON VARIOUS CLASSES OF ROADS

Item of cost	APPROXIMATE RELATIVE COST OF OPERATION, IN CENTS PER MILE		
	High-type roads	Intermediate-type roads	Low-type roads
Gasoline	1.09	1.31	1.61
Oil	0.22	0.22	0.22
Tires and tubes	0.29	0.64	0.84
Maintenance	1.43	1.72	2.11
Depreciation	1.26	1.39	1.57
License	0.14	0.14	0.14
Garage at $4 per month	0.44	0.44	0.44
Interest at 6%	0.36	0.36	0.36
Insurance	0.21	0.21	0.21
Total cost	5.44	6.43	7.50
Relative cost	1.00	1.18	1.38

In determining the allowance that should be made for decreased operation costs when a relocation shortens the distance between objectives, account must be taken of the fact that a shortening of distance does not affect the fixed charges incident to vehicle operation. A study of Table 1 would indicate that distance costs about 2.5 cents per mile on high-type surfaces and 3.3 cents per mile on intermediate-type surfaces, if it is assumed that one-third of the maintenance and depreciation is due to the mileage of travel and the remainder is due to weather and obsolescense. With commercial vehicles the shortening of distance also accomplishes a decrease in the time of travel between objectives, and it is believed that a reasonable allowance for the shortening of a high-type surface would be 8 cents per mile and for intermediate types, 13 cents per mile.

Analysis of Savings Resulting from Change of Road Surface Types

There is next presented an analysis of the change in cost of transportation resulting from changing the road surface on a section of U. S. Highway 65 from an ordinary gravel surface to a high-type paved surface. The traffic at the date of the change consisted of 700 000 automobiles per annum and a negligible amount of commercial vehicle traffic. The data used in this example are those on record for this particular highway, which is 26 miles in length. The investment data as of July, 1930, are first presented, and are followed by an analysis of the annual cost of transportation on this road while it was still a gravel surface.

As an intermediate type the investment in Highway 65 was, as follows:

Right of way	$38 000
Drainage structures	132 950
Earthwork	204 000
Road surfaces (light gravel)	84 700
Signs and other appurtenances	2 000
Engineering and administration, 4.5%	20 775
Total investment for 26 miles of road	$482 425

Annual Maintenance Cost.—The annual maintenance cost, including proper allowance for supervision, averaged $21 500 per year over a period of 5 years.

Periodic Maintenance.—The periodic maintenance consisted in re-shaping ditches and shoulders and re-graveling at intervals of three years at a cost of $34 300, which is E' of Equation (3). Since the time interval is so short it is really unnecessary to adopt the annuity method of determining the annual cost of periodical maintenance, but for illustrative purposes that method is followed herein. The annual cost of periodical maintenance is, therefore:

$$\frac{34\,300 \times 0.04}{(1.04)^3 - 1} = \frac{34\,300 \times 0.04}{1.125 - 1} = 34\,300 \times 0.32 = \$10\,980$$

Depreciation.—The total original investment in the gravel surface must be charged against the use of the gravel road since the salvage value of the gravel was zero when the high-type surface was laid. The depreciation charge is computed by the second term of Equation (3), which is, $\frac{(I - S)\,r}{(1 + r)^n - 1}$. In this case, $r = 0.04$ and $n = 8$ years. The annual cost of depreciation is, therefore,

$$\frac{84\,700 \times 0.04}{1.04^8 - 1} = \frac{84\,700 \times 0.04}{1.37 - 1} = 84\,700 \times 0.108 = \$9\,150$$

The value of A cannot be determined for the road in question. The annual road cost is determined by applying Equation (3), as follows:

$$C = 482\,425 \times 0.04 + \frac{84\,700 \times 0.04}{1.04^8 - 1} + 21\,500 + 10\,980 = \$60\,927$$

The length of the road is 26 miles and the annual cost per mile of road is. therefore, \$2 345.

The annual road cost after conversion to a high-type surface will next be computed. This road was improved with a pavement, signs were installed, guard fences were constructed, and the highway was completely modernized. As a high type the investment in Highway 65 was, as follows:

Right of way	\$38 000
Drainage structures	140 927
Earthwork	216 240
Prior surfaces (gravel having no salvage value)	0
Pavement	817 260
Signs and other appurtenances	4 450
Total	\$1 216 877

Engineering and administration costs were included in the foregoing items, as they were incurred.

Annual Maintenance Cost.—The annual maintenance cost has been determined from official reports to be an average of \$24 380 per year. This includes all the expenditures for maintenance of all elements of the highway.

Periodic Maintenance.—In this case, the need for any periodic maintenance has not occurred as yet. Based on the present forecast of probable life for this highway and the contemplated and feasible methods of prolonging the life of the investment, resurfacing of the concrete pavement may be effected at some future date, say, at the age of 20 years. At that time the existing road surface will serve as a base for a new surface and will have value as such. Assume that value to be \$1.50 per sq yd, or \$463 500, which is the factor, S, in Equation (3). The cost of resurfacing is forecast at \$275 000, which is the factor, E', of Equation (3). The value of n in Equation (3) is 20 years. The data are now assembled for the application of Equation (3) to the calculation of the annual cost of this highway, as follows:

$$C = 1\,216\,877 \times 0.04 + \frac{[(1\,216\,877) - (463\,500)]\,(0.04)}{(1.04)^{20} - 1} + 24\,380 + \frac{(275\,000)\,(0.04)}{(1.04)^{20} - 1}$$

in which, $C = 48\,675 + 25\,298 + 24\,380 + 9\,235 = \$107\,588$. The annual road cost per mile (length = 26 miles) is \$4 140.

The contribution to road funds, through the medium of the gas tax and license fee, by the automobiles using the gravel road, amounts to \$1 660 per mile of road per year and by those using the concrete highway it amounts to $1 570 per mile of road per year. The cost of transportation may now be computed from Equation (1) as follows:

I.—As a Gravel Road:

Comparative cost of transportation per vehicle-mile:

$$\frac{2340 + 45\,010 - 1\,660}{700\,000} = 6.53 \text{ cents}$$

II.—As a High-Type Road:

Comparative cost of transportation per vehicle-mile:

$$\frac{4\,140 + 38\,080 - 1\,570}{700\,000} = 5.80 \text{ cents}$$

The investment, therefore, is justifiable on the basis of the saving in transportation costs to the traffic actually using the highway.

ESTIMATING VALUE OF DECREASED DISTANCE

To illustrate the method of estimating the value of decreased distance the following example is presented. Suppose that the average annual traffic on a highway consists of 1 500 000 automobiles and 100 000 commercial vehicles, that a relocation is arranged that saves a distance of 0.6 mile, and that the pavement costs $25 000 per mile. The annual savings due to decreased distance would be, as follows:

Annual savings to automobiles = $1 500 000 × 0.025 × 0.6 = $22 500
Savings to commercial vehicles = 100 000 × 0.08 × 0.6 = 4 800
Total .... = $27 300

If it may be assumed that this relocation and consequent shortening of distance will be effective for the life assumed for the road surface, that is, 20 years, the present value of the annual saving of $27 300 is $371 000. To this would be added the $15 000 saving in cost of construction, making the present worth of the future service to be delivered by this relocation, $386 000.

Comparisons of this type should be used with great caution, however, because there is no assurance that the shortening of distance between two terminals, as was done in this case, will actually result in lowering the distance traveled by an automobile. To a considerable portion of the traffic there is this actual saving, but to another portion there is no such saving. It is a potential saving, however, and affords a means of forming a judgment as to whether a certain improvement is economically justifiable. In many cases the conditions surrounding a project are such that it is possible to determine with a considerable degree of accuracy the number of vehicles that will actually profit by the decreased distance.

GRADE REDUCTIONS

An exact determination of the economic value of reducing highway grades is impossible because of the heterogeneous character of traffic and the great variation between individual vehicles as to their performance on gradients and the lack of exact information with reference to that performance. However, there is some information available to aid the engineer in forming a correct judgment with reference to the expenditure that may be justifiable for a specific grade reduction project, and such additional data as may be required can be secured by a few simple field observations and at small cost. It is known that little saving in automobile operating costs is effected by reducing those grades that are less than 500 ft long, and that safety to descending traffic is a major consideration when grades are more than 1 000 ft long. In reducing the long grades to a safe rate there is usually a definite saving in operating costs, which can be estimated with some degree of assurance, after certain field data are at hand.

Minor changes in grade may be accomplished by cutting and filling without relocation, but major grade reductions, especially on long hills, are usually accomplished by a combination of relocation and cutting and filling. The changes in grade that may be accomplished by cutting and filling without relocation may or may not have economic value; in many cases they have none. There are three reasons why excessive grades may increase the cost of highway travel.

First, a round trip on a grade that is beyond the economical limit in rate (whether or not the vehicle must be operated in some gear other than high), results in an increase in fuel consumption over that required for traveling the same distance on the same surface on the level. If a vehicle is operated over various test courses, including level sections and various grades, the ratio of the gasoline required on the grades to that required on the level or on any other grade can be determined. In some studies made at Iowa State College this ratio (designated N herein) varied from about 1.1 to as much as 3.0. It will be taken herein at the lowest value likely to be found for grades ranging between 9 and 12% and the kind of traffic assumed for the illustrative problems that follow. This factor can be determined for specific grade reduction projects with sufficient accuracy by a few days of field work with an assortment of typical vehicles.

The second reason why excessive grades may increase the cost of highway travel is that energy is lost when there is necessity for controlling the vehicle with the brakes in descending a grade. It would be a simple matter to design a grade upon which a specific vehicle (say, the designer's own automobile) would coast at approximately constant speed or at gradually increasing speed, reaching the end of the gradient at approximately some pre-determined speed; but automobiles of some other design, especially of some other weight, would not perform in exactly the same way. For the greater proportion of the automobiles operating in high gear, with the throttle closed, the safe coasting grade is between 5 and 6 per cent. For commercial vehicles, especially the trailer combinations, the safe coasting grade is between 3 and 4 per cent.

The third reason is that the conventional method of driving is to operate the motor with the clutch engaged when descending a hill and, while thus operating, engine friction and the fuel pumped through the motor both become factors in the problem.

It is apparent, therefore, that highway grades must be a compromise between what is suitable for ascending vehicles and what is safe for descending vehicles. By a series of trials a grade can be fixed that is satisfactory from the operating standpoint and within the limits of the probable savings in the operating costs of the traffic.

The application of these considerations to a specific highway involves only a few rather simple computations and will be illustrated by a numerical problem for which the conditions have been assumed at random.

The physical conditions are set up as illustrated in Fig. 1, which shows a grade of an average rate of 12% and 2 000 ft long, which it is proposed to reduce to 7% by a relocation that requires the new grade to be 3 000 ft long. For the traffic on this road 4% is assumed to be the safe coasting grade; that is, on a 4% grade there would be some vehicles using a little power on the descent and some that would use the brakes, but the braking conditions would not be severe.

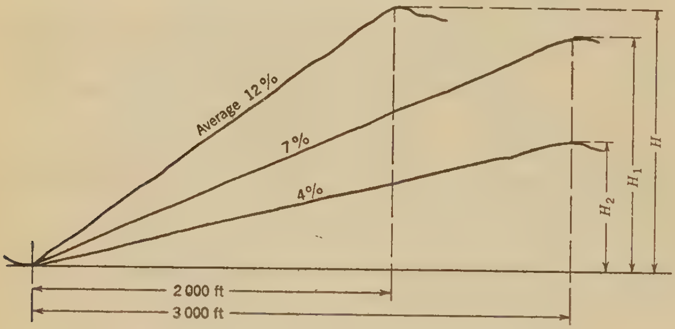

FIG. 1.—ILLUSTRATING ECONOMICS OF GRADE REDUCTION

For the purpose of these computations it may be assumed that a gallon of gasoline weighs 5.9 lb, tests 19 000 Btu per lb, and costs 19 cents per gal. Each British thermal unit is equivalent to 777.5 ft-lb, and the average efficiency of the automobile and truck engines in use is about 15 per cent.

The quantity of gasoline, in gallons, required to lift 1 ton 1 ft is:

$$G = \frac{2\,000}{19\,000 \times 5.9 \times 777.5 \times 0.15} = 0.00015$$

Assume the traffic on the highway at 4 000 000 tons per year, equally divided between the two directions of travel and consisting of 2 400 000 automobiles of average weight of 1.25 tons, and 250 000 commercial vehicles of average weight of 4 tons. For this problem, ascending grades are analyzed in four steps, as follows:

1.—The annual saving in cost to vehicles ascending the 7% grade as compared to the average 12% grade (neglecting increased distance for the moment) with $N = 1.75$, is,

$$S_1 = 0.19 \times 0.00015 \times 2\,000\,000\ [NH - H_1]$$
$$= 57\ [(1.75 \times 240) - 210] = \$11\,970$$

2.—Vehicles descending this grade need not use any power and the extra cost of travel for the 1 000 ft of added distance may be neglected since there are no power costs involved.

3.—The computations in Step 1 take no account of the cost of extra distance due to the relocation. There would be no extra power costs for the descending traffic, and the other items may be omitted. The extra costs per year for the ascending traffic will be equal to the cost of traveling 1 000 ft, which is, as follows:

For automobiles:

$$1\,200\,000 \times 0.025 \times \frac{1\,000}{5\,280} = \ldots\ldots\ldots\ldots \$5\,680$$

For commercial vehicles:

$$125\,000 \times 0.05 \times \frac{1\,000}{5\,280} = \ldots\ldots\ldots\ldots \underline{1\,180}$$

$$\$6\,860$$

4.—Summary for the ascending grades:

Total annual saving by the assumed grade reduction...	$11 970
Annual cost of extra distance.........................	6 860
Net annual saving..................................	$5 110

The conditions to the right of the summit shown in Fig. 1 will be considered in analyzing the descending grades, and, on this basis, the savings will be estimated as were those for the proposed ascending grades. Let it be assumed that for the minus grades, $H = 180$ ft, $H_1 = 150$ ft, the new minus grade is 7%, the old grade averages, 10.5%, and the new grade is secured by a relocation that does not increase the distance of travel on that side of the summit.

The saving per year to vehicles ascending the 7% grade as compared with the average 10.5% grade, with $N = 1.6$, is:

$$\begin{aligned} S_1 &= 0.19 \times 0.00015 \times 2\,000\,000\ [NH - H_1] \\ &= 57\ [(1.6 \times 180) - 150] = \$7\,866 \end{aligned}$$

The total annual saving by the grade reduction is:

On the plus grade..........................	$5 110
On the minus grade.........................	7 866
Total annual saving......................	$12 976

If the investment in grade reduction is to be amortized in 20 years by the annual payment of a sum equal to the estimated savings to present traffic, using an interest rate of 4%, the investment justified in this hypothetical case would be the present worth of an annuity of $12 976 running for 20 years, with interest at 4% compounded annually, which amounts to $176 348.

Similar computations for a grade of 5% instead of 7%, but with the summit 210 ft high, with distance increases of 2 200 ft and 850 ft on the plus and minus sides of the summit, respectively, will show a probable annual loss of about $1 100.

By way of contrast a similar computation may be made for a plus grade, 700 ft long, at an average rate of 10%, and, consequently, a rise of 70 ft, which is reduced to a 7% grade by cutting at the summit and filling in the sag. The new height will be 49 ft, and there will be no increase in the distance of travel. The factor, N, will be taken at 1.2. The annual saving to vehicles ascending the grade is $1 995.

A new factor now presents itself in that automobiles descending this grade (if the alignment is good) may be permitted to accelerate and thus utilize all the grade effect. The amount that can thus be utilized is the difference between the height of the 7% grade and one of 5% of the same length, amounting to 14 ft. The money value of the height thus conserved is $600 per year. The net annual saving by this grade reduction is, therefore, $1 995—$600, or $1 395. This would justify an investment of about $19 000 on the 20-yr annuity basis of amortization. The value of the grade reduction on the minus side of the summit may be estimated in a similar manner.

In all the foregoing discussion, it is assumed that all the vehicles are driven down grade in gear and with the clutch engaged, as is habitual with most drivers. If "free-wheeling" is used in descending the grades, the foregoing does not apply.

DISCUSSION

W. W. Crosby,[3] M. Am. Soc. C. E. (by letter).—A tenet, often advanced from certain quarters, is that the return in the form of economic transportation is the sole measure of justification for road improvement. Of particular interest is Professor Agg's apparent admission that this sordid tenet is not always to be accepted, but there may be some uncertainty as to just how far he means to go in emphasizing it. He states in the "Synopsis" that, excepting certain works to promote safety, most construction work of State Highway Departments and of cities consists of projects that should go forward only when they are justifiable on economic grounds. He states further that most of the major improvements do not result in a reduction of the annual cost of maintenance and must be considered on broader grounds, in order to justify the necessary expenditures. On the other hand he declares that in most cases the ultimate purpose of highway improvement is to permit transportation at the minimum cost.

Much seems to depend on just what Professor Agg intends the word "economic" to mean. As the writer attempted to suggest in a previous paper,[4] it may be used with varying meanings. Professor Agg seems to mean it to be interpreted narrowly as "saving." At the meeting of the Highway Division of the Society in Chicago, Ill., on June 29, 1933, E. J. Mehren, M. Am. Soc. C. E., pictured[5] the ideal highway system as one that is adequate for the traffic in each area served, properly marked, fitted with adequate signaling devices, with grade crossings eliminated, with footpaths, night lighting, landscaping, and with comfort stations. According to Mr. Mehren, engineers and other officials are projecting their minds into the future and preparing the way for safer, more comfortable, and less hampered traffic.

How can such a vision materialize if engineers are bound by acceptance of the illiberal dogma based on cold economics? Nevertheless, that materialization is yearly, if not daily, nearer reality in many places.

Charles B. Breed, M. Am. Soc. C. E., in discussing[6] the cost of highway transportation, has pleaded for a clear distinction between the cost and the value of transportation. Both factors must be appraised properly to determine whether the expense is justified. A highway, according to Professor Breed, may be entirely commercial, or it may serve partly for school transportation and for protection as well. Quoting:

> "It may be impossible to express these values in money, yet these may be so obvious as to carry unanimous public approval of a policy involving a road cost well above any saving that can be computed in dollars and cents."

Thus, according to the authority quoted, the justification of most large highway improvements will not be found entirely in their effect on transportation cost, but in their influence on transportation values, of which cost is only one element.

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[3] Cons. Engr., Coronado, Calif.

[4] "Economic and Engineering Problems of Highway Location," by W. W. Crosby, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 1007.

[5] *Civil Engineering*, August, 1933, p. 458; and *Roads and Streets*, August, 1933.

[6] *Proceedings*, Highway Research Board, Vol. 12, Pt. 1, p. 32.

Discussing a paper by the late Stephen R. Mather on "Engineering Applied to National Parks," H. K. Bishop, M. Am. Soc. C. E., likewise deplored[7] a tendency to think too much of economy in road construction and too little of the preservation of beauty.

The writer adds his endorsement to the foregoing idealistic concepts and submits that the results obtained by the Westchester County (New York State) Park Commission already emphasize the importance of certain "intangibles" connected with any highway improvement, which more and more demand consideration in questions of highway engineering.

The paper is based on logical reasoning within certain, perhaps narrow, monetary limits. One might dispute some of the author's assumed figures, and question the local acceptability of some of his "averages"; but probably this would lead to no results of real value, because of the neglect of the "intangibles" in the end. If he could evolve equations with the "intangibles" included, and a means for substituting "values" in them, highway planners might then welcome the general acceptance of a mathematical solution of some of the most troublesome of highway questions.

Roger L. Morrison,[8] M. Am. Soc. C. E. (by letter).—This paper is especially timely just now (1933) when those favoring "road-building holidays" are considering only the amount of their taxes, while most of those favoring more road building offer no reason for it except increased employment. Both sides practically ignore the important economic questions involved.

One reason for this, perhaps, is the necessarily complicated formula required in order to include all factors for an indefinite time into the future. Actually, of course, the most painstaking mathematical calculations are no more accurate than the basic data assumed, such as the life of the different parts of the highway and the ever changing costs of motor-vehicle operation.

It would seem, therefore, that reasonably accurate results might be obtained by considering only the period covered by the life of the first pavement. If this is assumed to be, say, 20 years, then the value of the highway at the end of that time can be estimated, and the difference between that value and the first cost is the depreciation, while one-twentieth of it is the annual depreciation. Periodic renewals of such parts of the highway as may not last 20 years may be allowed for in the sum estimated for annual maintenance. Under this method only the three items of depreciation, maintenance, and interest, need be considered in computing the annual cost.

It is freely admitted that this method is subject to attack from the standpoint of inaccuracy, and, also, that if an attempt is made to include in a formula all the operations involved, little may have been gained in the way of simplicity. For practical purposes, however, it does not seem necessary to include all these operations in a formula, and the method may possibly be more easily understandable, and hence more likely to be generally used, than the formal method given by Dean Agg. His accurate formula, however, is the standard by which any attempts at simplification should be judged.

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[7] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1193.

[8] Prof. of Highway Eng. and Highway Transport, Univ. of Michigan, Ann Arbor, Mich.

More serious than the differences between the various methods that may be used in making calculations is the scarcity of up-to-date information as to operating costs over various surfaces, particularly the so-called low-cost types which have been developed recently. With the enormous sums involved it would seem that the proper solution of these economic problems is sufficiently important to justify every highway department in collecting data on the cost of vehicle operation over different types of surfaces.

A word of caution appears to be desirable in connection with attempts to evaluate a highway system by means of an economic analysis. If an earth road, for instance, is replaced by a pavement, there may be an average saving of nearly 2 cents per mile for all vehicles that would use the earth road if the pavement were not there, but it does not follow that, where extensive traffic has developed after a system of pavements has been built (and largely because the pavements were available), there is a similar saving for all vehicles using the improved system. Without the pavements much of this travel would be by rail, or would not be undertaken at all, so that, to this traffic, "savings" indicated as computed herein, are largely imaginary. It is "proposed" highway expenditures which Dean Agg discusses, not expenditures several years in the past.

SAMUEL B. FOLK,[9] ASSOC. M. AM. SOC. C. E. (by letter).—This paper has done much to clarify current ideas, and to summarize a tangible method for attacking the problem of justifiable costs of improving a highway. It is somewhat unfortunate, however, that more emphasis was not placed on some parts of the analysis.

The basic principle of,

$$\text{Highway transportation costs} = \text{Highway costs} + \text{Vehicle costs}$$

should be somewhat elaborated and qualified by provisions that these costs shall fall equitably on those able and willing to pay; that collections must be made promptly; and, that payments must be sufficient to retire outstanding obligations when due. At times, there is a tendency to refer to the cost per ton-mile or per vehicle-mile that would justify improvement in the aggregate, omitting from consideration the painful fact that a few constant and important users of the new road will profit by decreased vehicle-operating costs, while many occasional users save little in operating costs and are required to meet the heavier interest and amortization charges on bonds retired from taxes on abutting or on adjacent property. This is an unfortunate but archaic system followed under some State laws; and while this phase of highway economics is beyond the scope of Dean Agg's paper, it is hoped that engineers will rise to sponsor an equitable method of financing improvements as well as a justifiable program.

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[9] Asst. Prof. of Mechanics, Ohio State Univ., Columbus, Ohio.

The assumption that from 2.5 to 3.3 cents per mile would be reasonable for an allowance due to shortening of distance seems fair and is borne out by many studies.[10] The same may be said for the 0.8 cent per automobile mile saved in changing from one type of surface to the next higher. However, current notions of economics have undergone such serious upsets in recent years that it might be well to apply a "factor of safety" to any calculation involving the saving per car-mile multiplied by the total traffic per year. Such studies usually assume a constant annual traffic that is probably nearly correct for commercial vehicles; but a lower type of roadway discourages pleasure driving and thereby affords a real saving, whereas a higher type of surface encourages more pleasure driving (which operates at a lower cost) than a poor road. It has not been shown from studies, however, what percentage of traffic is chargeable to business and what percentage to pleasure, which would otherwise not be undertaken and which, therefore, cannot be credited to savings. The same philosophy might be applied to the saving derived by shortened distance as with savings accruing from operation on higher types of roads. Dean Agg has done well in pointing out the ease of arriving at such a conclusion by fallacious adaptation of the results of careful investigations.

W. S. Downs,[11] M. Am. Soc. C. E. (by letter).—The economic value of proposed highway expenditures merits more consideration than the subject has generally received during the hasty construction of road systems in the United States. Indications are that future highway improvements will receive more careful analyses to determine the economic justification of such expenditures, in order that the tax dollar, which has become more difficult to produce, will accomplish the greatest possible benefit. As a result of his careful research and studies in highway economics, Dean Agg has repeatedly pointed the way to sound principles that are an aid to the highway engineer in solving such problems.

His method of determining the annual cost of highways is generally accepted, although there is some confusion concerning the propriety of including an annual interest charge on the public investment when such funds are derived from taxes. Assuming that the motoring public has paid a substantial part of the capital cost of the highways, or that the capital investment comes from general funds raised by taxation, to which the motor-vehicle owner contributes, the question is asked: "Should the motor-vehicle owner be charged interest on funds which he himself contributes?" Dean Agg takes the position, correctly, that proper account of interest must be introduced if strictly correct conclusions are to be drawn His explanation in this respect is satisfying.

Care must be taken in evaluating Equation (3), not to place too great values on S, the salvage value, and n, the number of years the improvement will serve economically. Experience has demonstrated that the economic life expectancy of highway improvements may be easily over-estimated; not so much on account of the enduring qualities of the structures as to the changing needs of traffic. Many improvements have become obsolete by reason of necessary changes in location or design due to the increased speed or in the weight and size of the vehicles.

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[10] *Bulletin 91*, Iowa Eng. Experiment Station, Ames, Iowa.

[11] Cons. Engr., Morgantown, W. Va.

Likewise, the forecast of traffic should be made conservatively. Annual traffic surveys, with comparatively few exceptions, show a pronounced flattening of the curve of increase within recent years. Many important highways show considerable decreases in the use of the roads. That such trend is not due wholly to a temporary decrease in automobile use is indicated by the fact that it was noticeable in many sections as early as 1928 and 1929, during very prosperous years. It is probably the natural sequence of the extensive increase in the mileage of improved highways, which permits the traffic to distribute itself more widely over parallel routes. The approach to a condition of congestion on any highway will cause the traffic to seek other routes quite as much as a poorly paved surface, bad alignment, or greater distance. The liquidity with which traffic seeks the route of least resistance to its convenient flow makes uncertain the forecast that it will continue to use the highway under consideration when, and if, other competing roads are improved.

Since so many factors enter into the cost of vehicle-operation it would appear impossible to derive a formula that will serve for all highway improvements. Dean Agg more or less confines his discussion to such factors as may have a direct bearing upon vehicle-operation costs, as they relate to various kinds of road surfaces. For this purpose he uses the road classifications, "high type, intermediate type, and low type," to express comparatively the physical characteristics of the surface. It is unfortunate that a more definite classification cannot be suggested that would better define the surface properties that make for economy of vehicle operation. The all-important element in this respect would seem to be a smooth, firm surface that will lower the tractive resistance. Perhaps there are others; but roughometer tests will indicate that the surface of the so-called intermediate and low-type highways may often be as smooth as those that are generally considered to belong to the high type.

There are other elements of highway improvement, however, quite as important in determining vehicle operation costs as the type of surface. Alignment, as well as grade, requires consideration. Poor alignment not only creates accident hazards, but it reduces the speed of vehicles. The grade, likewise, not only affects gasoline consumption and such other factors of cost related to the mechanical operation of vehicles, but it also reduces the speed of traffic. This suggests a careful investigation of the time element as it may influence the time value of the driver and the occupants of vehicles and as it may be the contributing cause of added mechanical costs due to the less efficient speeds of the faster moving vehicles.

The average present-day passenger car may negotiate a grade of 7 or 8% in direct drive with no appreciable loss of speed, and, consequently, it experiences little loss of efficiency on such grades. The heavy truck, however, may find it necessary to change to a lower gear ratio on grades of 3 or 4 per cent. In addition to the economic loss suffered by the truck due to the gear change, the slower speed with which it travels imposes on all other traffic on the highway the necessity of either passing such vehicle or reducing the speed to that of the truck. If the alignment is such as to restrict sight distance, or if intervals between vehicles approaching in the opposite direction are not great, the passing of a slow-moving vehicle (which is generally of excessive length), becomes impossible. The loss of car-time, therefore, and the necessity for operating at inefficient speeds are ineconomies imposed upon all traffic.

A remedy for this condition may lie in the construction of four-lane highways which will permit the segregation of the slow-moving vehicles to the outer lanes, thus reserving the inner lanes to the faster moving cars. This, however, entails highway costs that can only be justified economically by a great volume of traffic, such as does not, at present, exist on many highways.

In most studies involving the economics of highway expenditures, it is necessary to consider the motor vehicles as divided into two classes: First, that of the private passenger car, and vehicles of like weight and size which possess common characteristics (this class is sometimes referred to as the "basic type"); and, second, that class composed of the so-called commercial vehicles which vary widely as to size, weight, and engine performance.

In his studies, Dean Agg considers the automobile (or basic type) to include all commercial vehicles with a capacity of 1 ton or less. The commercial class, he considers, is made up of commercial vehicles having capacities of 1 ton or more.

It is difficult to co-ordinate these classes to highways in common use. Highway improvements that are necessary for the efficient operation of one class of vehicles may scarcely benefit the other class; or, the benefits to be derived by one class can be measured only by the removal of restrictions that have been imposed by the presence of the other class on the highways.

It is questionable, therefore, if in determining the economic value of certain highway improvements, it is proper to assume that all classes of vehicles will share alike in the benefits of such improvements either on the basis of the vehicle-mile or the ton-mile. Moreover, the wide differences in the requirements of the two classes of vehicles and the variation in the character of the traffic in different localities precludes such studies being made on the presumption of an average vehicle.

This suggests the desirability of research and study in the economy of providing distinct and separate highways for the two major classes of vehicles, especially where the volume of commercial traffic would appear sufficient to justify such steps.

The basic automobile traffic requires easy curvature, but grade reduction is a secondary consideration. The strength of the pavement will be in proportion to the unit wheel load, and its width (if not reduced from present standards), will offer more convenience and ease of travel. The commercial traffic requires less refinement in the highway alignment, but demands easy grades. The strength of the pavement, likewise, must be in proportion to the heavier wheel load.

A final thought in connection with this subject relates to the financing of the proposed highway improvements. It is not enough to show that the required expenditures are economically justified in that the savings in vehicle operation costs will exceed the cost of the improvements. Further consideration is required as to whether or not such savings can be capitalized to finance the improvements. Notwithstanding the economic value of highways, expressed as savings in vehicle operation costs, it remains a fact that motor vehicle imposts, thus far, fall far short of paying all the costs of the highways. Highway funds have been (and still are, in part), derived from other sources, such as special assessments and property taxes. The assessment of part of the cost to properties may generally be justified for city streets and roads of little general use, on the theory that special benefits accrue to the property owners; but if the improvements are made solely for the benefit of the highway users in the sense that a low-type surface is replaced with a higher type, or that grade reductions or alignment changes are made in order to reduce vehicle operation costs further, then it is extremely difficult to show that additional benefits will accrue to property owners. When property is taxed for purposes from which it derives no benefit, the value of such property depreciates. This practice is detrimental to the entire tax system, since it subsidizes one class at the expense of another.

Hence, as a final test of the economic value of such improvements to the State, assurance is needed that sufficient additional revenues will be derived from the owners of vehicles, who profit from the improvements, to compensate the State for such cost.

George E. Martin,[12] Assoc. M. Am. Soc. C. E. (by letter).—The method of obtaining the comparative cost of highway transportation per vehicle-mile on a specified highway, as developed by Dean Agg in this paper, is probably correct, but the use of the old cost data (see Table 1) originally published in 1928,[2] may lead to serious errors when applied to present-day road surfaces.

The relative cost of operating an imaginary "average" automobile on various classes of road is shown in Table 1. The highway engineer who attempts to use this table will substitute for the terms, "high type," or "intermediate type," or "low type," the particular road he has in mind. The first and last terms are fairly clear, but what is to be included in the "intermediate type"? Dean Agg does not define this term in the paper, although elsewhere[3] he has defined roads as follows: (1) High types (pavements of all kinds in average good condition); (2) intermediate types (gravel, macadam, and bituminous-treated surfaces); and (3) low types (natural soil roads and light gravel or sand-clay surfaces).

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[12] Cons. Engr., The Barrett Co., New York, N. Y.

On the basis of these definitions, engineers have often assumed that the cost given under "intermediate types" (Table 1) would apply to all roads not specifically included in the other two classes. This assumption is not correct. As a matter of fact, these values apply to gravel roads and not to bituminous surfaces or to the mixed-in-place types of road surface. Many of the data in Table 1 were collected in Iowa and practically all the roads in that State, except city pavements, prior to 1928 were earth, gravel, or concrete. Nearly all the test runs were made on these three types, and the writer has been able to find no published data prior to 1928 on bituminous surface treatment, and only a few on bituminous macadam.

The operating costs shown to vary between types are gasoline, maintenance, depreciation, and tires and tubes. Dean Agg has made the statement[13] that maintenance cost is assumed to vary with fuel consumption and one-half the depreciation is also assumed to vary with fuel consumption, the remainder being due to obsolescence. Assuming that the cost of fuel is 1.00 for the high-type pavements, Dean Agg gives the comparative fuel consumption for the intermediate type and the low type as 1.20 and 1.47 units, respectively. These are the constants he has used in preparing Table 1 of the paper.[14] Relative fuel consumption, therefore, is the basis for these varying costs, and to check the supporting data, it is necessary to refer to a previous paper[15] by Dean Agg and Harold S. Carter, M. Am. Soc. C. E., in which the relative (yearly average) costs of vehicle operation on roadway surfaces of average Portland cement concrete and ordinary gravel are given (see Table 2, Column (3)). These costs check exactly with the relative cost factor, 1.18, for the intermediate types of roadways in Table 1 of the paper. Dean Agg and Professor Carter[16] also present a tabulation that gives the relative fuel

TABLE 2.—DATA PERTAINING TO OPERATING COST

Item No. (1)	Type of pavement (2)	Cost of operating automobiles at from 25 to 35 miles per hour, in cents (3)	Relative fuel consumption, cars with pneumatic tires, operating at 35 miles per hour (4)	GASOLINE CONSUMPTION		TIRE WEAR, IN POUNDS PER 1 000 MILES		
				In miles per gallon (5)	Comparative value (6)	McNown (7)	Dana, 1924–1925 (8)	Dana, 1926 (9)
1	Concrete	10.00	1.12	14.2	1.00	1.43	0.093	0.097
2	Gravel	11.80	1.36	13.1	1.08	0.78	....	......
3	Bituminous-treated gravel	.....	......	13.9	1.02	....	....	......
4	Oil-treated crushed rock	.....	......	......	......	....	....	0.027
5	Crushed basalt macadam	.....	......	......	......	....	0.476	......
6	Crushed rock macadam	.....	......	......	......	....	....	0.372

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[13] *Bulletin 91*, Eng. Experiment Station, Iowa, State Coll., Ames, Iowa, July 25, 1928, p. 17.

[14] *Loc. cit.*, Table 5.

[15] "Highway Transportation Costs," by T. R. Agg and H. S. Carter, *Bulletin 69*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, July 9, 1924, p. 20.

[16] *Loc. cit.*, p. 19.

consumption on road surfaces of good Portland cement concrete and gravel, the latter approximated from the Iowa yearly average (see Table 2, Column (4)). This produces comparative values for concrete and gravel roads of 1.00 and 1.22, respectively, which check quite closely with the high-type and intermediate-type values cited previously herein.

Table 1 was evidently correct for gravel in 1924 when the data were collected, but a quite different ratio between concrete and bituminous-treated gravel can be deduced from the results reported by R. G. Paustian, Jun. Am. Soc. C. E., in 1932.[17] The Highway Research Board has published curves[18] which show the effect of road surface on gasoline consumption. Interpolating from these curves for a speed of 35 miles per hr, the results in Table 2, Columns (5) and (6), are obtained for good dry concrete, dry gravel, and bituminous-treated gravel, surfaces.

This would indicate that the values in Table 1 based on a constant of 1.20 between the high type and the intermediate type of surface are certainly not correct for a comparison between concrete and bituminous surface treatments.

The other variable cost is that of tire wear which, according to Dean Agg,[15] is assumed as 1.00 for high-type roads, 2.12 for intermediate types, and 2.90 for low types. Data on this subject have been published by William C. McNown, Assoc. M. Am. Soc. C. E.[19] (see Table 2, Column (7)). Professor H. J. Dana, of the State College of Washington, reports[20] the average results listed in Table 2, Column (8). These averages are for tests run in 1924, 1925, and 1926. One test on oil-treated macadam was run in 1926 and is reported[21] as shown in Table 2, Column (9).

Nothing in Table 2 would indicate a differential in tire wear between concrete and bituminous surface treatments of 2.12. The writer believes that available data are insufficient to assume that the values in Table 1 for "intermediate types" can properly be applied to road surfaces which the average highway engineer considers intermediate types, such as bituminous surface-treated gravel and mixed-in-place surfaces. The values in Table 1 are not averages for the entire range of intermediate types, but apply only to plain gravel. Additional data will be necessary before true values for these intermediate types can be obtained, and while these data are being collected, it is manifestly unfair to apply the values of Table 1 to intermediate types other than plain gravel.

J. T. L. McNew,[22] M. Am. Soc. C. E. (by letter).—The author has long been giving advice to the profession as to the economic aspects of highway improvement, but only in recent years have any road users allowed the engineer to improve roads only to the extent directed by such economics.

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[17] *Proceedings*, Highway Research Board, December 1–2, 1932.

[18] *Proceedings*, Twelfth Annual Meeting, Highway Research Board, p. 87.

[19] *Proceedings*, Sixth Annual Meeting, Highway Research Board, 1926, Table 1, p. 65.

[20] *Engineering Bulletin No. 18*, State Coll. of Washington.

[21] *Loc. cit.*, p. 14.

[22] Prof. of Highway Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

As long as road users' demands were insistent that more miles of pavement be laid, and they did not complain of the tax burden, few were the engineers who would direct attention to such subjects as "justifiable expenditure," "savings in transportation costs," etc.

With the débacle of 1929 and its effect upon tax payments, the diversion of road toll levies for poor relief, the talk of road-building holidays, and the passing of all bond markets, the stage has been re-arranged and the scenery changed to cast the remainder of the road construction projects on a more sound financial procedure.

Perhaps the slowness with which highway economics has proceeded has been due, in a measure, to the slowness with which a National road-building program was undertaken. The accentuated demand for public roads began about 1900, when the automobile made its début, but it was about sixteen years later before the United States embarked upon a program of Federal roads. By 1916 the demand for roads was so insistent that it could not wait and for the next thirteen years nothing mattered much as long as the roads were paved, and the sooner the better. A perusal of typical publications of about 1908 will show that first the public had to be "sold" on the idea that good roads were a benefit and, thereafter, the people bought wholeheartedly and unstintingly.

With about two years now (1932 and 1933) in which to pause and reflect on the need for roads, one must quickly see that no longer can the public bear expenditures on capital improvements unless the tax levied and collected will be returned to the taxpayer either in whole or in increased amounts. Perhaps the financial return may be slightly less at times than the tax burden, and the increment of difference will represent an elevation of the standard of living; however, communities that recognize such a principle are few in this day when standards of living are apparently below the peak of former years.

Perhaps the alacrity with which the public has provided road construction funds may be due to the typical American custom of buying a thing, and then dismissing the matter. In other words, the public has been permitted to think that once a road was paved it would be paved permanently and, with such a thought, the cost price was provided. Now that the probable life cycle of every type of pavement has been about reached or passed since the beginning of road policies in the United States, it is about time all engineers "took a hand," and showed the economic facts that justify road improvement and refuse to countenance programs not based upon such facts.

Practically all expressions for annual cost involve the term, residual value or salvage value, for instance, indicating that one stage of construction may well be a valuable part of the next type of improvement, and yet this one factor is often overlooked in so-called Unit-1 construction to such an extent that when a road type is changed from a gravel surface to a higher type, it becomes necessary to change the grade so much that the salvage value of the gravel becomes zero. In such cases, unless traffic type has changed radically, there is an indication of forgotten economy. In recent years the researches of the U. S. Bureau of Public Roads have shown that certain sub-grade soils, namely, Classes A-6 and A-7[23] are probably unsuited as sub-grades under high types of pavements and that the sand-clay and gravel mixtures are capable of providing valuable sub-grades. It is probably true, then, that by planning road improvements more than one step ahead the engineer may actually reduce the loss on each stage of construction and, at the same time, have each stage justified.

The author's analysis of the change in cost of transportation on United States Highway 65, resulting from a change from a gravel to a high type of surface, offers an example of the point the writer desires to make. Under the cost analysis of the gravel surface, the author shows the entire value of the surface to be charged against the gravel surface and that as a sub-grade for the higher type the old gravel has zero value. Is not this treatment indicative of either an ill-chosen grade line for Unit 1, or a rather severe charge to the gravel type? If it is assumed that all the gravel is bladed to the shoulders, is not then the traffic-compacted sub-grade of some value in excess of that of the freshly constructed sub-grade, and would not the value of the gravel in the shoulders be the equivalent of a measurable salvage value? In many cases the writer feels that the salvage value of an old surface, when used as a foundation for a high type of pavement, may be even more than the original surface cost, under certain conditions.

The economics of highway location with respect to distance saved or to the cost of operation over excess distance is another phase of the general problem that has caused much debate. Capitalizing theoretical savings of transportation due to shortening of distance offers many pitfalls, and yet it is a logical part of any cost analysis. The writer doubts the wisdom of setting up figures to justify savings of distance unless those figures are chosen with conservatism. Is an engineer justified in assuming that any average road, located with reasonable care and skill, will for all time to come be the right location, or that it will offer the same service as it does when it is built? Unless the answer is in the affirmative, one may say that the justification for relocation may change and when that change is demanded the savings will no longer pay the interest on the first cost of the improvement, especially in cases where the amount first computed as justifiable actually has been spent. The writer recognizes that the cost of extra distance may be tremendously large in some cases, but he doubts very much if the user of the road who actually accumulates the savings appreciates the fact, and he also doubts that the ones who do accumulate the savings would ever be willing to donate even one-fourth of such savings back into construction funds as a road tax.

Dean Agg has computed the justifiable expenditure for saving distance by first assuming that the saving will be effective for twenty years. This is a noteworthy change from the methods used to justify saving distance as often published. Twenty years may be a long time in numerous cases, and the writer agrees with the author that "comparisons of this type should be used with great caution.

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[23] *Public Roads*, Vol. 12, No. 5, July, 1931, p. 136.

H. E. PHELPS,[24] M. AM. SOC. C. E. (by letter).—Old data relating to the cost of vehicle operation cannot be used unchanged in economic comparisons of various types of surfaces, when one of the surfaces is either bituminous treated gravel, mixed-in-place crushed stone surface, or of the plant-mixed type. These surfaces are "intermediate" as regards cost, but all available evidence places them very close to pavements as regards vehicle-operating costs.

This evidence falls into two classes, dealing with gasoline consumption and tire wear. As to gasoline the latest evidence is given by R. J. Paustian, Jun. Am. Soc. C. E.[25] From Fig. 2 it may be concluded: (*a*) That the roughness of concrete surfaces has no appreciable effect on gasoline consumption; (*b*) that bituminous treated gravel is better than wet concrete for speeds less than 38 miles per hr; (*c*) that at a road speed of 40 miles per hr bituminous treated gravel has a fuel consumption of 1.06 (approximately) as compared with concrete (wet or dry), reducing to an inappreciable difference at speeds between 25 and 30 miles per hr (as compared with 1.20 in Table 1); and (*d*) that bituminous treated gravel is much more closely a high type than an intermediate type as regards fuel consumption.

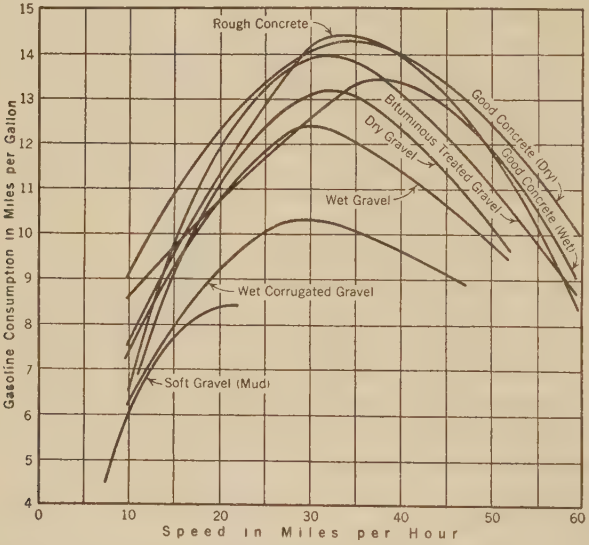

FIG. 2.—EFFECT OF ROAD SURFACE ON GASOLINE CONSUMPTION

There is little real evidence that tire costs are any greater on bituminous treated surfaces than on concrete. The cost of tires as an item of vehicle-operating costs has no very direct relationship to the item of tire wear, since accidents and age rather than wear frequently necessitate tire replacements. Tire wear tests at the State College of Washington do show greatly increased wear on untreated gravel surfaces consisting of sharp-edged basaltic rocks, but they also indicate very strongly that tire wear on bituminous, surface-treated gravel roads is at least no greater than on concrete.

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[24] Prof., Highway Eng., State Coll. of Washington, Pullman, Wash.

[25] *Civil Engineering* July, 1933, p. 377, Fig. 2.

A. A. Anderson, Assoc. M. Am. Soc. C. E., and Mr. H. B. Wright have reported[26] 81% greater tire wear on non-skid asphaltic concrete than on concrete, but W. C. McNown, M. Am. Soc. C. E., reports[27] that tire wear is nearly the same on all firm-surface roads. This evidence indicates that tire wear is much more closely related to the texture of the surface than to either its roughness or its kind, and that file-like surfaces such as exist on some of the open non-skid bituminous mixtures or on Portland cement finish by brooming or wiping will produce greater tire wear than the more smoothly traveled surfaces of earlier concrete pavements. There is no evidence that modern oiled roads or asphaltic concrete surfaces cause either greater or less tire wear than modern concrete pavements, and until this evidence is forthcoming it is futile to attempt to draw valid conclusions as to the relative economic merits of different types of pavements.

C. C. WILEY,[28] M. AM. SOC. C. E. (by letter).—The economics of highway expenditures is a problem that is both elusive and confusing. The fundamental concepts are so entangled with governmental processes, sociological requirements, and financial operations that it is almost impossible to trace them, while many of the factors necessary to a solution are as yet unknown and perhaps even impossible of determination. Economic writers seem to have severed this Gordian knot by the simple expedient of waving aside all but the financial aspects, and then to have treated these aspects by conventional methods. The conclusions reached may therefore be far from reliable.

The present-day discussion of highway economics seems to have started about twenty-five years ago when the U. S. Bureau of Public Roads, in its efforts to stimulate interest in better highways, pointed out that the capitalized value of the decrease in hauling costs due to an improved surface would generally be more than the cost of the improvement and, therefore, the many benefits of good roads, both social and economic, were, in effect, to be had without additional expense.

From that simple beginning there appears to have evolved a current theory of highway economics based on the two major assumptions: (1) That road improvements must be justified by reduced transportation costs; and (2) that such costs are to be computed as if the entire expenditures for road construction were capital investments in a commercial enterprise for profit.

The first assumption is in error because it makes use of only one of several economic benefits of good roads and entirely ignores the social advantages. It is true that some writers admit that there are other than financial considerations, but the general inference, whether intentional or not, is that they are relatively unimportant. Other writers, however, state positively that the improvements must be justified by reduced costs of transportation.[29]

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[26] Rept., 11th Annual Meeting, Highway Research Board.

[27] Rept., 6th Annual Meeting, Highway Research Board.

[28] Assoc. Prof., Highway Eng., Univ. of Illinois, Urbana, Ill.

[29] "Highway Construction, Administration, and Finance," by E. W. James, M. Am. Soc. C. E., Highway Education Board, Washington, D. C.

This assumption is approximately correct, however, when it is preceded by, or based on, the additional assumption that each type of road under consideration can render the required basic service. Costs of transportation then become the distinguishing feature between the various types. If the various kinds of roads can not render identical service then all advantages must be considered and the social benefits may become the controlling factor, with hauling costs the least important in the selection of the type to use. The point that seems to have been lost is that if reduced transportation costs are sufficient to finance the improvement, all the other benefits are obtained for nothing. The mere fact that the savings in hauling costs will not pay for the improvement does not necessarily make the improvement a luxury.[29] The proper assumption is that, other things being equal, the improvement should be justified by reduced transportation costs and then extreme care must be taken to see that the "other things" actually are equal.

The second assumption seems to be the outgrowth of borrowing the ordinary formula (the author's Equation (2)) for theoretical investments without regard to the actual conditions that surround the financing of road improvements by governmental bodies. The net result is that this assumption is fundamentally wrong.

Highway expenditures are in no sense capital investments. They are the direct cash outlay by the people for the purchase of a service perhaps as fundamental to present civilization as government itself. If, however, some improvement in the service results in a saving to those receiving it then this saving, and this saving only, may be considered as available for a limited time to pay for the improvement. It must be remembered, however, that the cost of a road improvement (including interest on bonds, if any), is actually paid out by the people in the form of taxes, while the savings accrue later by the reduced costs in the purchase of other items incidental to the use of the road.

Neither the governmental units that build the roads nor the taxpayers who pay for them, can be considered as investors in the usual commercial sense. Government is essentially a "corporation not for profit"; and hence it is not an investing body. To accept the author's statement that "had that sum remained in the hands of the individuals who comprise the public, each might have invested his share in a dividend or interest paying security," is to establish the principle that capital interest can be charged on tax payments. This is in direct violation of all business practices whereby taxes are invariably written off as items of expense. The entire theory of computing highway costs as capital investments fails on this one point alone.

As further evidence, it would be absolutely essential, in order to make the theory tenable, that each and every taxpayer would have equal and ample facilities for the safe, easy, and adequate investment of his funds. No such possibility exists and probably never could exist. What a "thrifty" few may do under existing conditions can not be assumed to be what all could do under other conditions. Government or road bonds could not be depended upon since there are not enough of them. When one person buys a bond some one else must sell; hence, there would have to be enough new bonds each year to absorb the road funds for that year. Consequently, recourse would have to be made to other kinds of investments with all their variable characteristics of safety, convenience, and adequacy of return. The only sane way to invest tax funds is to pay the taxes, and no other investment yields as great a return, using the term, "investment," in its broad sense.

The commercial method of computing highway costs has the further fault that it is based on the use of annuities. Past experience with accumulations of public funds have been so consistently disastrous that their use is rapidly disappearing. Sinking-fund transactions by public bodies are specifically barred by law in some States. It seems to be a growing principle of taxation that taxes shall be levied only for current expenditures (including debts due) and not for accumulation. It is scarcely logical to base computations on a type of transactions that legally can not be made.

Incidentally, the item for depreciation in the cost formula (Equation (2)), has been confusing to many people. The attitude taken is that the road has been paid for, and to charge depreciation is making them pay for it twice. This is not correct, of course, because the depreciation item is merely the device for including the first cost; but instead of including the actual cost a fictitious amount in the form of an annuity is introduced.

The proper consideration of bond interest has been confusing only because the true function of the bond issue has been overlooked. Road bonds are used when it is desired to complete a definite system in a short time so that the generation paying the bill can have the use of the entire system while paying for it. Thus, a county that needs 200 miles of improved roads for adequate service to the people, but can build only 10 miles per year from current funds, would require 20 years to build the system and at no time during this period would the entire mileage be available for use. If, however, bonds were issued and the roads were built in 2 years, the entire system would be available for 18 of the 20 years. To do this would require the payment of interest which, therefore, would become merely a premium paid for the privilege of having the system now instead of twenty years from now. Bond interest, therefore, is simply an added item of first cost and should appear as such in computing either the total or the annual costs.[29]

The proper calculation of highway costs should be based only on the actual sums involved. The equation for average annual cost of a road improvement, therefore, becomes,

$$C = \frac{I - S + i + M + A}{N} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which, C is the average annual cost at the age of N years; N, the age, in years, over which the cost is distributed; I, the initial cost of the project, including financing, construction, engineering, and administration; S, the salvage value at the age of N years; i, the total interest paid in the period, N years; M, the total maintenance cost in the period, N years, including special or periodic repairs; and, A, the total administrative and other costs during the period, N years.

The investment idea persists in computing vehicle costs in the form of an item for annual interest. This practice is probably correct when the vehicles are essential items in the capital investment for the equipment of a commercial enterprise, such as a road contractor or trucking company. It is questionable, however, when the vehicles are merely substituted for some other form of transportation expense (as would be the case when a business concern supplies its salesmen with cars to replace railroad travel, and which is probably the correct status of the majority of private cars). If the car was purchased on deferred payments, the financing charges would have to be included. This item of interest, however, is not large, and since the costs are based on broad averages from which there is a wide range of variation, it probably does not seriously affect the accuracy of the vehicle costs per mile.

The two examples of highway costs given by Dean Agg are worth a little study. In the case of the gravel road, the total cost includes all items, and this is also probably true of the maintenance charge since no contrary statement is made. The 3-year periodic repairs would probably be made by dividing the 26-mile line into three sections, and repairing one section each year from current funds at a cost of $440 per mile, instead of accumulating a sufficient fund every 3 years by an annuity of $422 per mile. Possibly, the money would be borrowed, which would add an interest item to the cost. Depreciation appears to be computed only on the gravel surface; hence, the other items included in the total cost must be presumed not to depreciate.

Turning, now, to the concrete road it is found that, again, total costs and maintenance costs cover all items. Periodic repairs, however, are considered as occurring only once in twenty years and then in the form of a new surface on the old concrete as a base. The sum allowed for this purpose obviously will not replace the surface with concrete; hence the pavement has now changed type. It is no longer a concrete pavement; consequently, the original "investment" has ended and a new computation based on the new type must be started, just as when the change was made from gravel to concrete.

Depreciation has been computed on a sum found by subtracting the salvage value of the concrete surface from the total cost of the road. Therefore, it includes 100% depreciation on earthwork, right of way, etc., which obviously is not correct. In both cases the life of these items should be estimated, and the cost prorated over the term considered.

Using Equation (4) and including, for simplicity, only the items relating to the surface, the cost of the gravel road is,

$$C = \frac{84\,700 + 20\,775 - 0 + 0 + 20 \times 21\,500 + 6 \times 34\,300}{20} = \$37\,064$$

or $1 425 per mile. In the case of the concrete road,

$$C = \frac{817\,260 - 463\,500 + 0 + 20 \times 24\,380}{20} = \$42\,068$$

or $1 618 per mile.

These values combined with the given vehicle costs yield 6.40 and 5.45 cents, respectively, for the comparative transportation costs on the gravel and concrete. This gives a difference of 0.95 cent per vehicle-mile in favor of the concrete. Considering this difference to apply to the entire 700 000 vehicles per year for the full 26 miles there results an annual saving of $172 900. This is the annual sum that may be considered investable in the improvement because it actually remains in the hands of the people.

In computing such investments it is customary to consider the annual saving as an annuity and to determine its present value for a given term at a given interest rate. Since annuity bonds are little used, and since in many States serial bonds are required by law, it would seem more logical to determine the amount of serial bonds on which the annual saving would equal the maximum annual payment of principle and interest. This amount is given by the equation,

$$P = \frac{S}{\frac{1}{N} + r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

With $r = 0.04$ and $N = 20$, the amount of bonds which the annual saving of $172 900 can handle, is $1 921 111, instead of $2 349 767 as given by the annuity method. This is the sum which, theoretically, can be borrowed to change the road from gravel to concrete. Since the cost of 26 miles of concrete was only $817 250, it is evident that the improvement is justified. Since the total cost was only $1 216 877, the saving will pay for the entire road which is more than it is required to do. By using $P =$ $817 250 and solving for N it is found that the saving would pay for the change from gravel to concrete in 7 years with 4% serial bonds.

The examples of decreased distance and grade reduction both illustrate the investment of the savings resulting from improvements. The first is valuable because of the figures presented for the cost of short changes of distance. The second is an excellent brief outline of the plan of attack of the grade-reduction problem. Great care is needed in any particular case in analyzing the conditions and in evaluating the factors to be used.

The only other point which needs consideration is the item of time. The value of the time factor has been greatly exaggerated in all discussions of road and street economics. Small periods of time are valueless because they can not be used. Thus, the saving of 1 min or 2 min to each vehicle on a given road is no economic gain, because this small increment of time can not be put to use. There is a vast difference in the economic value of the delay of 1 min to each of 60 cars and the delay of 60 min to 1 car, although the "car-minutes" are the same. The customary practice of multiplying these small fractions of time by the large number of people or vehicles to which they accrue individually, and assigning a value to the product, is erroneous. It is only when the fractions of time to some individual become of usable size that they become of value, and then only to that individual. Just what minimum period of time begins to have an economic value is unknown. It is probably different for every location and for every vehicle that passes that location, It must be quite evident to even a casual reader that the economics of highway improvements are not, as yet, established. Many factors must be evaluated, and the entire problem must be carefully analyzed. There is a fine field for further work in this line. Meanwhile, highway engineers should not take any theory or data too seriously lest they deceive themselves and mislead the public.

THOMAS R. AGG,[30] M. AM. SOC. C. E. (by letter).—The discussion of this paper has been gratifying and has brought out some divergent ideas which are of value in clarifying this somewhat illusive subject. Perhaps the writer's ideas as to the place of analyses of this type in highway planning was not entirely clear. It is when the engineer has before him several alternate plans and wishes to determine which is best from the economic standpoint (purely money comparison), that he must undertake some such analysis as that discussed. Frequently the somewhat alluring proposal is wholly without justification upon any utilitarian basis. It may be justifiable on some other grounds, but such is not usually the case with the ordinary run of projects of a highway department.

The lack of vital basic data for economic analyses was pointed out by several of those who discussed the paper, and it is to be regretted that so little research is under way in this field. Better information than that now available with reference to highway and vehicle costs, the useful life of roadway surfaces, and related subjects, should be forthcoming. A better classification of highways for the purposes of economic studies would also be of great benefit.

With reference to the use of interest and annuities in building up annual costs for economic comparisons, it is believed that if these factors are neglected wholly misleading results will be obtained. Of course, it is not pretended that interest is paid or that sinking funds are accumulated, but intermittent expenditures of varying sums cannot be placed on a comparative basis without taking account of the time value of money. In principle there is no difference between a public project and a private enterprise in this respect, or at least there should be no difference in the treatment of costs if one wishes a true picture of the relative merits of several possible solutions to a highway problem. If one were planning a financial program for a political unit, the problem would be wholly different.

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THE SURVEYOR AND HIS LEGAL EQUIPMENT

By A. H. HOLT,[1] M. Am. Soc. C. E.

With Discussion by Messrs. Ray H. Skelton, William Bowie, R. Robinson Rowe, Walter H. Dunlap, Chester Mueller, Clarence T. Johnston, A. F. Harley, Lynn Perry, C. H. Eiffert, Verne G. Sanders, and A. H. Holt.

Synopsis

There are many places at which the work of the Engineer and that of the Lawyer overlap, but possibly there is no department of his professional activities in which the Civil Engineer has more specific need to use either his own or another's knowledge of the law than in land surveying. Especially is this true in the re-establishment of lost or disputed boundaries.

To some persons a complete answer to the problem presented by a lost or disputed boundary seems to be to call in a surveyor to "run the line." If a "starting point" is available, even if it is remote from the boundary to be located, a common impression seems to be that a competent surveyor should have no uncertainty in how to proceed, nor experience any real difficulty in arriving at a definite solution. In fact, the result is scarcely to be thought of as a "solution." The surveyor should be able, it seems, by the use of his instruments alone, to establish the boundary with certainty, and with assurance that, thus established, it will stand. A larger group would probably be willing to assert that when the dimensions—courses and distances—of a boundary are clearly evident, perhaps on a plat, any consideration of the location of the boundary by other means than the careful running out of the description on the ground would be idle.

If through earlier mistake or carelessness that which may long have been accepted as the boundary does not agree with that which is now precisely located, the sooner the change is made the better. This attitude toward their

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abilities and powers may possibly be complimentary to surveyors; but to attempt to measure up to its demands a surveyor must either be unencumbered by the possession of a conscience, or unhampered by experience.

It is desired to discuss two aspects of boundary location—very briefly its importance, and somewhat more at length some of the factors which control or influence it. Those who are interested in studying this general subject further are referred to books written by the late Ray H. Skelton,[2] M. Am. Soc. C. E., and Mr. Frank E. Clark.[3]

A Fundamental Proposition

A fundamental proposition should be stated at the outset: That the proper location of a boundary is that which a competent and well-advised Court would decree. The only exception to this rule is a location on which the owners concerned may agree for themselves. This they may always do validly, as long as no other interests are thereby jeopardized.

That the proper location of a boundary is a matter of some consequence, that failure in this respect can be as costly as shortcomings in engineering design, the decisions of the Courts of last resort of the United States are ample testimony. In the system of law that governs this country, drawn from its heritage of feudal customs and traditions, fostered by its development through a period in which, it has been said, the law of England was framed to help the King of England run his farm, no property rights are given a higher grade of protection than is given to rights in land. The name, "real property", is not without meaning.

One of the most fundamental of these rights is that of exclusive possession. If one of two co-terminous owners, because of an erroneous conclusion as to the location of the common boundary, extends his building on to his neighbor's land, there is nothing left for him to do, if the neighbor cares to insist, but to remove it and, in addition, to pay damages for the intrusion. He cannot, through his own wrong (the trespass) force his neighbor to sell the strip of land to him. True it is, that in a very few cases an Equity Court has restrained the execution of a judgment of ejectment, or has refused an injunction; but only when the action of the invader has appeared in a favorable light, and it has been shown that there is no substantial damage to the use of the land invaded. One trial Court, for example, finding that the trespass was to the extent of ½ in. beyond the boundary, refused to require the removal to that extent of an underground concrete foundation. A higher Court, however, reviewing the evidence on appeal, concluded that the intrusion was over a strip about 3 in. wide, and decreed its removal. The cost of complying would have paid, many times over, for an accurate survey.

A prominent building in New York City was found to have been extended beyond the line. The owners, in this case, found it possible to persuade the adjacent owner to sell to them the 6-in. strip involved. The cost of the persuasion is stated to have been $50 000. This was less than the cost of removing the building, but an accurate survey would have saved it.[4]

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[2] "The Legal Elements of Boundaries and Adjacent Properties," by Ray H. Skelton. The Bobbs-Merrill Co., 1930.

[3] "A Treatise on the Law of Surveying and Boundaries," by F. E. Clark, The Bobbs-Merrill Co., 1922.

The difficulty in each of these cases arose from lack of appropriate precision in the execution of a survey. It is unnecessary to discuss the impossibility of an exact survey; and a Court cannot do otherwise than accept a reasonably precise location in lieu of one as exact as the language of lawyers and of Courts might seem to require. Is it sufficient, then, for the safeguarding of the interests of clients, that a surveyor be capable of making surveys accurately and with all the precision which is reasonable and appropriate for the property concerned? The answer of an experienced and honest surveyor to this question would probably be that he can do the surveying well enough if he can only make certain what should be done. It is this determination of what factors should influence or control the location of the boundary, which requires a surveyor, in order to render to his client service of a maximum value, first to function as a counsellor-at-law—perhaps it is not too much to say, in some respects, as a Court.

Without in any way minimizing the importance of precise surveying, it may be said that in the re-establishment of uncertain or disputed boundaries the use of surveying instruments or calculations is but one means—although an extremely valuable means—of assembling evidence, or of giving effect to a decision, as to where the line should be.

The comment may perhaps be made that a surveyor should be concerned only with surveying—not with interpretation of evidence or application of law—that his activities should be restricted to a survey and a report to his client, leaving the client to do with them as he thinks best. A surveyor who adhered to this narrow path would be a tradesman indeed. As a practical matter, the client usually expects the surveyor to determine where the line should go and depends upon him to do so. The client does not desire a report which will require him to seek the counsel of a lawyer as to how to proceed. Thus, it is that in order to render to his client the service that the occasion frequently demands, a surveyor, called in to "run a line," may need to possess and use a sound knowledge of the law of boundaries. Lacking this knowledge, or lacking experience and judgment in its application, he may easily become a cause of trouble between adjoining owners, instead of fulfilling his proper function of justly and judicially disposing of the difficulty.

By a misguided survey, even though it may be an accurate one, he may aggravate to the point of Court action a situation which with proper handling would have been settled amicably and economically; or, if the case be one that must be settled in the Courts, he may bring his client into Court on an improper, and therefore a losing, ground. The writer is acquainted with more than one case in which a mutual and friendly desire on the part of adjoining land-owners to have their common boundary properly located was turned into a dispute by a needless or an inaccurate survey of the line by the surveyor. In one such instance the proper location was finally reached only after four surveys, much hard feeling, an erroneous decision in the District Court, and a reversal by the Supreme Court of the State. Two days' work by a competent surveyor at the beginning should, and probably would, have ended the matter.[5]

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[4] Mulrein *v.* Weisbecker, 37 App. Div. 545, 56 N. Y. S., 240; Wachstein *v.* Christopher, 128 Ga. 229, 57 S. E. 511, 11 L. R. A. (N. S.), 917; Murphy *v.* Bolger Bros., 60 Vt. 723, 1 L. R. A. 309, 15 Atl., 365. However, see Harrington *v.* McCarthy, 169 Mass. 492, 48 N. E., 278; Methodist Episcopal Soc. *v.* Akers, 46 N. E. 381, 167 Mass., 560; Rahn *v.* Milwaukee Elec. Ry. & Lt. Co., 103 Wis. 467, 79 N. W., 747.

A CARDINAL PRINCIPLE

There is one cardinal principle of the law of boundaries, applicable to this situation, which should never be disregarded, namely:

> A boundary line having once been authoritatively—even though inaccurately, or even erroneously—fixed in position on the ground, the true purpose of a re-survey is to reproduce that location; not to "correct," or otherwise change, it.[6]

No statute, no incumbency of office, can give to any person the right to "establish" a line, by precise survey or otherwise, in contravention of this principle, and to the disturbance of existing rights. If a boundary has once been authoritatively established, property rights have been vested in some one with respect thereto; and the mere obliteration or loss of the physical evidences or even of the record of the boundary, cannot divest those rights. Furthermore, a more precise re-running of the survey by which a boundary was once located cannot detract from, or add to, the property rights that have already accrued, and thus shift an established boundary.[7] From the more precise survey a more accurate description may perhaps be written, but the original boundary must stand.

ORDER OF CALLS

A surveyor's first duty, then, when called in to re-establish a boundary of which the location has become uncertain, or is in dispute, is the judicial task of assembling and evaluating the various available items of evidence of its original position. Unfortunately, there is likely to be some conflict or disagreement among these items, and a proper relative weight or importance must be assigned to them. This precedence of certain types of evidence of original boundary location is known as the "order of calls."[8] It must be continually kept in mind that the purpose to be accomplished is to recover the boundary as it was located originally; or, if it was not actually marked on the ground, then to carry out the intention, as effectively expressed, of those by whom or for whom the boundary was originally established or described. For this reason, those evidences of boundary which are most material, most certain, and least liable to error should prevail.[9] Any order of calls or other rules must be used and interpreted so as to further this purpose in the case in hand. They should never be applied arbitrarily when such application would obviously defeat it.[10]

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[5] Meyers *v.* Wonick, 180 Ia., 286.

[6] Nesselrode *v.* Parish, 59 Ia. 570, 13 N. W., 746.

[7] Diehl *v.* Zanger, 39 Mich., 601.

[8] Note, 129 Am. St. Rep., 990.

[9] Newsom *v.* Pryor, 7 Wheat (20 U. S.), 7, 5 L. Ed., 382.

[10] Moran *v.* Lezotte, 54 Mich. 83, 19 N. W., 757, 759; Connor *v.* Johnson, 59 S. Car. 115, 37 S. E., 240; Pritchard *v.* Rebori, 135 Tenn. 328, 186 S. W. 121.

Experience has shown that marks on the land itself, being capable of being seen by owners, and not being subject to errors of survey, computation, or description, are most likely to express the intent of parties. The lines and corners actually marked upon the ground by the parties concerned—by the owners of adjoining tracts in establishing a boundary, or by a grantor in connection with a conveyance—will prevail, if they can be recovered, over all other calls, even over conflicting calls for more substantial monuments, to which the description may be referenced.[11] It is essential, however, that these marks of line or corner, in order to have this effect, be so named in the description of the deed or other document as to make it apparent that they actually expressed the intended location at the time it became effective.[12] This rule will prevail, also, when the discrepancy is with reference to a call for an adjoining boundary to which the survey is said in the description to run, if the survey was actually run and marked on the ground without reaching the named adjoining boundary.[13]

In case of conflict between marks placed upon the ground to indicate boundaries, and maps to which deed descriptions may refer for location of the same boundaries, the rule seems to be that "it is by the work as executed on the ground, not as projected before execution, or represented on the plan afterwards, that the actual boundaries are determined."[14] Although a deed may call for a plat only, not mentioning any survey—for instance, in transferring a numbered lot as shown on a certain plat—it has been held that a call for a lot according to a plat is a call for that which is only recorded on or represented by the plat, namely, the boundaries as marked in the field.[15] Marks of boundary will control over conflicting descriptions in terms of course and distance, and also over any call in a deed for a specified quantity of land conveyed.[16]

To be thus preferred these marks of boundary must have been made prior to or at the time of the grant, and it must be apparent that they were adopted as such (by reference in the deed, or otherwise) by the parties concerned; and the marks now found must be identified as those to which the reference is made, or else they cannot be regarded as expressing the intention of the parties.[17]

Recognizing that the boundaries actually marked by or for the parties concerned are to control if they exist and can be found and identified, and remembering that this and all other rules concerning evidence of the location of boundaries exist only for the purpose of discovering the intent of parties as effectively expressed, there remains the necessity of considering the relative value or effect of the various classes of evidence of boundary. The marks of boundary just mentioned will naturally fall into one or another of these classes. In this order of preference, natural monuments are given first place; artificial monuments are second; calls for adjoiners rank third; courses and distances, fourth; and quantity or area inclosed, last.[18]

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[11] Jackson *v.* Miller, 195 S. W. (Mo.), 703; Thatcher *v.* Matthews, 101 Tex., 122, 105 S. W., 317; Messer *v.* Reginnitter, 32 Iowa, 312.

[12] Ritter Lbr. Co. *v.* Montvale Lbr. Co., 169 N. Car., 80, 85 S. E., 438, 443; Elliott *v.* Jefferson, 133 N. Car., 207, 45 S. E. 558, 64 L. R. A., 135.

[13] Matheny *v.* Allen, 63 W. Va., 443, 60 S. E. 407, 129 Am. St., 984.

[14] Ovens *v.* Davidson, 10 U. C. C. P., 302, 310; Rowell *v.* Clark, 119 Iowa, 299, 93 N. W., 280.

[15] Jackson *v.* Freer, 17 Johns. (N. Y.), 29, 31.

[16] Millerman *v.* Megaritty, 242 S. W. (Tex.), 757, 760; Arms *v.* Owatonna, 117 Minn., 20, 134 N. W., 298.

[17] Woodbury *v.* Venia, 114 Mich. 251, 72 N. W., 189; M. K. & T. R. R. Co. *v.* Anderson, 36 Tex. Civ. App. 121, 81 S. W., 781, 784; Currier *v.* Jones, 121 Iowa 160, 96 N. W., 766.

Any physical landmarks adopted or established to indicate a boundary are regarded as "monuments."[19] While the classification of monuments as "natural" or "artificial" distinguishes, generally, works of Nature from those of Man, the purpose of the classification apparently causes some variation from this line of distinction. Obviously, streams, shore lines, trees, etc., are natural monuments;[20] and such structures as walls, well-built fences, and highways, although they are works of Man, have been so classed. The following definition is on record:[21]

> "Natural monuments are objects permanent in character, which are found upon the land as they were placed by Nature, such as streams, lakes, and ponds, shores and beaches, also highways and streets, walls, fences, trees, hedges, springs, and rocks, and the like. * * * Wherever natural monuments, referred to in the description of a deed as marking the boundary, can be found, they fix the limits of the premises granted, though they correspond neither with the courses and distances nor with the quantity of the land given in the same descriptions."

Artificial monuments, for the purpose of assigning relative weight as evidence of boundary, are objects of lesser stability and permanence, or objects set specifically to mark points of boundary, such as stones, concrete markers, iron pipes or pins, etc.[22] Wooden stakes have been rejected by some Courts as monuments,[23] but if substantial they are usually acceptable.[24]

Calls for adjoiners are in a sense calls for monuments in that they require the land to extend to a line which exists, or may be brought into existence, on the ground, independently of the survey or description under consideration.[25] As such they are generally to be preferred to course and distance.[26] However, a large discrepancy might indicate a mistake in naming the adjoiner, or a misunderstanding as to where the line of the adjoining tract was, so that the preference would be reversed, especially if the called boundary is an unmarked line.[27]

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[18] Meyer *v.* Comegys, 147 La. 851, 86 So., 307; Yanish *v.* Tarbox, 49 Minn. 268, 51 N. W., 1051.

[19] Koch *v.* Gordon, 231 Mo. 645, 133 S. W., 609, 610.

[20] Miles Land Co. *v.* Hudson Coal Co., 246 Pa. 11, 91 Atl., 1061.

[21] An American and English Encyclopedia of Law, Second Edition, p. 764.

[22] Sayers *v.* Lyons, 10 Iowa, 249.

[23] Bast *v.* Mason, 165 Mo. App. 718, 726, 148 S. W., 398; Cox *v.* Freedley, 33 Pa. 124, 130, 75 Am. Dec., 584.

[24] Tomlinson *v.* Golden, 138 N. W. (Ia.), 448; Thatcher *v.* Matthews, 101 Tex. 122, 105 S. W., 317.

[25] Bryant *v.* Maine Cent. Ry. Co., 79 Me. 312, 9 Atl., 736; Fidelity Realty Co. *v.* Flahaven Land Co., 193 Ky. 355, 236 S. W., 260.

[26] Land Co. *v.* Saunders, 103 U. S., 316; Graybeal *v.* Powers, 76 No. Car., 66, 69.

[27] Miller *v.* Grunsky, 141 Cal. 441, 75 Pac., 48; Jones *v.* Hamilton, 137 Ky. 253, 125 S. W., 695.

Monuments and lines of adjoiners are actually boundaries, while courses and distances are at best only descriptions of boundaries. This being the case, these descriptions, or courses and distances, can control only (1) when there are no calls for actual boundaries; (2) when some mistake precludes the use of the actual boundaries which are specified; or (3) when these actual boundaries have become lost, not merely obliterated. Courses and distances are valuable to assist in finding or in identifying marks of boundary;[28] and they themselves may determine the boundary when no superior call appears, or when the superior call is found to be erroneous.[29] In the case of an inconsistency between course and distance some Courts appear to favor allowing the course to control. The following opinion[30] by the United States Supreme Court seems the better solution:

> "It may be laid down as a universal rule that course and distance yield to natural and ascertained objects. But where these are wanting, and the course and distance cannot be reconciled, there is no universal rule that obliges us to prefer the one to the other. Cases may exist in which the one or the other may be preferred upon a minute examination of all the circumstances."

The distinction between a "lost corner" and an "obliterated corner," as the terms are used herein, is a fairly clear one. The reference is really to the physical object which once marked the point in question. If that object has disappeared, but if the position which it once occupied can be determined by any reliable extrinsic evidence (such as bearing trees, fence lines, or even the testimony of persons who have seen it), the corner is only obliterated, not lost; and it then has all the controlling effect that it would have had if the physical marker had remained. If not only the physical marker which once stood at the corner is gone, but there is insufficient reliable evidence to point out with reasonable precision the place it occupied, the corner is lost; and its control is gone. The arbitrary measures of surveying sometimes prescribed for the relocation of corners can apply only to situations in which the corners have become lost—never if they are merely obliterated.

Any corner which is "re-established" by any arbitrary method of surveying—such as, for example, a quarter-section corner re-established by marking the mid-point of a straight line joining the two adjacent section corners—is quite likely to occupy a position materially different from that originally marked. This new location, although it may conform to the requirements of statute, can have no effect in divesting any private rights of land ownership which can be shown to exist, even if the boundaries of such ownership were originally dependent upon the corner thus "re-established." Only in cases where the sole available control for such boundaries consists of descriptions dependent upon the surveyed line can this newly established corner determine old private boundaries.

Unless the quantity of land conveyed by a deed is made the essence of the contract, or unless there is a covenant to convey a specified amount of land,[31] the statement in the deed of the area conveyed has little effect in controlling boundaries:

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[28] Palmer *v.* Magers, 85 W. Va. 415, 102 S. E., 100, 103.

[29] Andrews *v.* Pitts, 126 Md. 328, 95 Atl., 203.

[30] Preston's Heirs *v.* Bowmar, 6 Wheat, (19 U. S.) 580, 582, 5 L. Ed., 336.

> "A definite description of lands in a deed designating the initial point and the courses and distances, followed by a statement of the number of acres conveyed, passes the quantity of land embraced in the specified boundaries, though greater or less than the number of acres stated."[32]

The words, "more or less," frequently follow the statement of quantity. Their purpose should be to show that all the land embraced within the specified boundaries is intended to pass by the conveyance, even though the actual area be somewhat at variance with that stated. However, these words should not so qualify the statement of the deed as to throw all risk of deficiency,[33] upon the grantee. In one case the grantee was granted relief where the deficiency was approximately 8 per cent.[34]

The foregoing suggests some of the principles of law that bear upon the recovery of an uncertain boundary. Their effect is first to require a careful assembling of all available items of evidence of the original location. Of prime importance in this process is an exhaustive search on the ground for the "footsteps of the surveyor," and for any other physical evidences of boundary. This should be no haphazard, superficial inspection; but a careful, methodical search. Consideration of this matter is quite outside the province of this paper; but any surveyor of experience is acquainted with instances of the futility of the former, and of the occasional success of the latter under seemingly hopeless conditions.

All evidence disclosed must then be weighed, in the light of the law as developed by the decisions, to ascertain the most probable location of the original lines. This law is simply a judicial expression of the results of long experience in working toward this purpose. The "running of the line"—that means by which the surveyor is frequently expected at once to produce a definite conclusion—will be done only in furtherance of and in conformity with this preliminary judicial consideration. To save time by a more hasty process is sometimes both disappointing and expensive.

DOCTRINE OF ADVERSE POSSESSION

The necessity may appear, however, of first determining whether the original boundary should or need be recovered at all; whether, on the other hand, it may be that by some development over a period of years another line may, through action of law, have been substituted for that which was once the boundary. One of two adjoining owners may occupy and use land beyond the line which surveys and deed descriptions would indicate as his boundary. If this is done in a way and for a length of time to satisfy the conditions of the law, the public interest in permitting him to remain peaceably in the situation in which he has established himself will induce the

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[31] Sanders *v.* Godding, 45 Iowa, 463, 465; Doyle *v.* Mellen, 15 R. I. 523, 8 Atl., 709.

[32] Syllabus by the Court, Pohlman *v.* Evangelical Lutheran Trinity Church, 60 Neb. 364, 83 N. W., 201; Dashiel *v.* Harshman, 113 Iowa 283, 85 N. W., 85, 87.

[33] Darling *v.* Osborne, 51 Vt., 148.

[34] Paine *v.* Upton, 87 N. Y. 327, 41 Am. Rep. 371.

Courts to decide that he has acquired a prescriptive title to the land. The title of the old owner is then gone, because a new title has come into existence which is incompatible with the old. Had the old owner acted in time he could have prevented the acquisition of this new title, and thus have retained his own; but it is now too late. A common boundary has thus been fixed by occupation, replacing the original one; and to re-run surveys or to attempt to execute the deed descriptions on the ground would be idle and of no avail. This is the American doctrine of "adverse possession"—a doctrine of affirmative prescription. In Iowa, this doctrine is almost inseparably connected with that of "acquiescence."

The decision as to whether prescriptive title has been acquired by adverse possession, or whether the conduct of the adjoining owners has by acquiescence established a boundary, is, in most cases, for the Courts; but it is essential that a surveyor be cognizant of these possibilities, and that he be acquainted with, and able to recognize, the items of evidence which tend to establish the existence of either of these situations. Thus, he may avoid a useless survey, and will be able to assist in assembling the evidence which will enable his client's attorney, or if necessary, a Court, to reach a proper decision.

The attitudes of the Courts in these matters have developed quite differently in the different States, and it will be best to limit this paper to a brief and general consideration of the law of one jurisdiction. Hence that which follows is based particularly on the law of the State of Iowa. A study of the cases is essential, of course, to any accurate or detailed understanding of the law of any jurisdiction.

To culminate in a prescriptive acquisition of title, it is necessary that the possession of the land by the invader be (1) actual, not merely constructive; (2) open and notorious; (3) under color of title or claim of right; (4) exclusive; (5) hostile to the interests to which the possession is alleged to be adverse; and (6) continuous for the period of the statute of limitations. In Iowa, this period is ten years.[35] Many pages of the reports of the decisions of American Courts have been occupied with defining these requirements, and determining whether the facts of specific cases measured up to them. Two or three of the cases in Iowa are outstanding in their development of the Iowa law regarding the situation with which surveyors are most concerned; that is, where the occupation has been by some one who has acted in good faith, with no purpose of acquiring land which did not belong to him, but who, because of mistake or misunderstanding, has for more than ten years occupied land beyond what now appears to be the surveyed or described boundary.

Doctrine of Acquiescence

An early case[36] established the position that the Iowa Court has since maintained, namely, that the belief of the possessor that he owns the land possessed does not amount to or take the place of the claim of right required by law. The possession under mistake is held not hostile to the interests of the true owner, because the possessor intended to occupy only that which belonged to him; and, consequently, it is not "adverse." That would appear to dispose of the matter in many cases in which a surveyor might be concerned, leaving the ownership in the holder of the paper title; but a decision by Mr. Justice Ladd in 1900,[37] while it cited and re-affirmed the decision in the case of Grube *v.* Wells, laid down another closely allied doctrine which, in Iowa, supplements, almost to the point of superseding, the doctrine of adverse possession. That is the doctrine of acquiescence. In that case two adjoining owners had for many years occupied and cultivated up to a certain fence, treating it as their common boundary. A survey developed the fact that the fence was several feet from the surveyed boundary, which was a line of the Public Land Surveys. The Court stated:

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[35] Code of Iowa, 1931, Section 11007.

[36] Grube *v.* Wells, 34 Iowa, 148.

> "But, if it be conceded that this survey tended to fix the location of the government line as originally established, it does not follow that it should be regarded as the boundary between the coterminous owners. * * * But if the coterminous owners have adopted another line as their division line, and have occupied up to it and recognized it as such for a period of ten years, there apears to be no reason for not regarding it as the true boundary line, notwithstanding it is not that fixed by the government survey. * * * In the absence of other controlling circumstances, the inference is conclusive that the division line between adjoining tracts, definitely marked by the erection and maintenance of a fence or other monuments, recognized by the owners as such, and up to which they have occupied and cultivated the land on either side for more than ten years—the statutory period of limitations—is the true boundary between them."[38]

This decision is in accordance with a statute which was then Section 4236 of the Code of 1897, and is now Section 12306 of the Code of Iowa, 1931. Thus, it is seen that on the ground of adverse possession, or on that of acquiescence, boundaries between private owners may, by usage alone, be established in a fence or in a line otherwise marked. It is immaterial whether this line is that for which deed descriptions might call, or that which a retracing of Government or private surveys might locate; hence, the matter is one for consideration before time is spent in search or surveying. If a line has thus been fixed, a new survey may sometimes be profitably made to perpetuate it, but a re-survey cannot change it.

Unfortunately, the cases are not altogether clear in the matter of distinguishing adverse possession from acquiesence. It has been held that boundary lines may be established by recognition and acquiesence, independently of adverse possession, except for the element of ten years of time;[39] and that although the doctrine of adverse possession does not apply, in Iowa, to a case in which a party has had no intent to claim more than his deed calls for, a line may be established by recognition and acquiesence, although neither of the parties has intended to claim more than is described in his deed.[40]

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[37] Miller *v.* Mills County, 111 Iowa 654, 82 N. W., 1038.

[38] Miller *v.* Mills County, 111 Iowa, 654, 657, 659, 82 N. W., 1038, and cases there cited.

[39] Helmick *v.* D., R. I. & N. W. R. Co., 174 Iowa 558, 156 N. W., 736.

[40] Bradley *v.* Burkhart, 139 Iowa 323, 115 N. W., 597.

It has been held that one who pleads that a certain line is the true boundary because it has been recognized and acquiesced in for the statutory period, must stand or fall on that theory; that he will not, after failing to establish acquiescence, be permitted to prevail on the claim of adverse possession only.[41] Other cases, however, hold that one who for ten years has claimed title to and occupied premises up to a definite boundary line with the apparent acquiescence of the adjoining land-owner has acquired title either by acquiescence or else by adverse possession.[42] With this matter of distinction, however—important though it is to the lawyer and to the Court—the surveyor need not ordinarily be concerned.

From the standpoint of identifying the situation in which establishment of boundary by acquiesence may be the result, it is important to note the requisite acts, attitude, or knowledge on the part of the claimant (of land beyond the surveyed boundary) and on the part of the land-owner against whom the claim is made. Although the doctrine is often referred to as one of mutual acquiesence,[43] the acquiesence is, in reality, all on the part of the owner adjoining the claimant. The claimant must show, by clear and definite acts,[44] an intention to claim to a definite line[45] as the true one, which he must honestly believe it to be.[46] The mere establishing of a line,[47] or mere occupancy, is not sufficient. If this be acquiescence it is of a very positive character, and is very similar to that open and notorious possession required for adverse possession.

The acquiesence of the adjoining land-owner, on the other hand, may be of a passive, negative character.[48] It may be shown by his conduct, as by occupying up to the line[49]—although that is not required[50]—by words, or by remaining silent when he had notice or knowledge of existing conditions.[51] It has been held, that where the land encroached upon is vacant and the owner absent, the doctrine is inoperative,[52] on the ground that actual knowledge on the part of the adjoining land-owner is essential to his acquiesence.[53] On the other hand, it has also been held[54] that the erection by the claimant of a house on a city lot, with reference to the survey stakes there existing, was notice to the world of her claim as to the location of the boundary, so that the adjoining owner and his non-resident grantors were held to have acquiesced in the boundary as such.

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[41] Morrow *v.* Hall, 169 Iowa 534, 151 N. W., 482.

[42] Dake *v.* Ward, 168 Iowa 118, 150 N. W., 50.

[43] Taylor *v.* Olmstead, 201 Iowa 760, 206 N. W., 88.

[44] Griffin *v.* Werdell, 182 Iowa 969, 164 N. W., 760; Lynch *v.* Northwestern Laundry, 194 Iowa 317, 189 N. W., 748.

[45] Quinn *v.* Baage, 138 Iowa 426, 114 N. W., 205.

[46] Keller *v.* Harrison, 139 Iowa 383, 116 N. W., 327.

[47] Webster *v.* Shrine Temple Co., 141 Iowa 325, 117 N. W., 665.

[48] Dake *v.* Ward, *supra.*

[49] Morley *v.* Murphy, 179 Iowa 853, 162 N. W., 63.

[50] Kotze *v.* Sullivan, 231 N. W. (Ia.), 339.

[51] C. M. & St. P. Ry. Co. *v.* Hanken, 140 Iowa 372, 118 N. W., 527; Dwight *v.* City of Des Moines, *infra.*

[52] Palmer *v.* Osborne, 115 Iowa 714, 87 N. W., 712.

[53] Dwight *v.* City of Des Moines, 174 Iowa 178, 156 N. W., 336, 339; Morley *v.* Murphy, *supra.*

[54] Helberg *v.* Kepler, 178 Iowa 354, 159 N. W., 972.

It is generally held that the doctrines of adverse possession and acquiescence do not apply to land held for public purposes by the Federal[55] or State Governments, or by municipalities.[56] A private owner who fails for a long period of time to exercise his right to eject an invader, finds that that right has been cut off. The public can act only through its agents, the elected or appointed officers; and public interests should not be permitted to suffer through their unauthorized neglect. It may be noted in passing that if, through positive acts of a qualified representative of a municipality, a private owner is led to occupy land beyond his proper line, and into that which should rightfully be devoted to public use, the doctrine of equitable estoppel may come into play to prevent the municipality from so disavowing the acts of its representatives as to require the private owner to withdraw.[57]

ESTABLISHMENT OF BOUNDARIES BY AGREEMENT

Acquiesence does not presuppose an agreement to a line; but, on the other hand, agreement to the boundary is to be inferred from long acquiesence therein.[58] However, the establishment of a boundary by agreement may often be the most economical, most sensible, and most satisfactory means of settling the location of an uncertain or a disputed boundary. Authority that such an agreement, properly consummated, will be effective and binding may be found both in the decisions and in the statutes.[59] It is not intended that this process of agreement on a boundary should be used to effect a change in the position of a known boundary, thus resulting in the transfer of land without the formality of a deed;[60] although it has been held that an oral agreement to change a long established boundary is enforceable when taken out of the statute of frauds by (1) the mutual taking of a new survey; (2) the building of a new fence in accordance with the said survey; and (3) the taking possession of the lands inclosed by such fence.[61]

It frequently happens, however, that a surveyor is called upon to relocate a line for which the control is difficult to find, expensive to reach, and of very doubtful certainty or precision. The general location of the line is usually known to the adjoining owners. If they would forego the survey, and establish the line by compromise and agreement, or permit the surveyor as an arbitrator to make such a compromise location, the loss to either owner in value of land involved between a location thus made and the extreme position for which he might contend, would frequently be far less than the cost of the search and survey. The compromise location having been agreed upon, the line may be monumented, surveyed, tied in to enduring control, and platted. This procedure, followed by a formal and recorded acceptance of the line by adjoining owners as their boundary, will fix that line as such for all time. The saving in dollars is often considerable, but the saving in good will is the most valuable result. Recommending this procedure to his client, explaining it to the client and to adjoining owners, and assisting in its consummation, is in some cases the most valuable service that a surveyor can render.

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[55] Stanley *v.* Schwalby, 147 U. S. 508, 514, 37 L. Ed. 259, 13 Sup. Ct., 418.

[56] Schultz *v.* Stringer, 168 Iowa, 675; Quinn *v.* Baage, 138 Iowa, 426, 114 N. W., 205; Johnson *v.* City of Shenandoah, 153 Iowa 493, 133 N. W., 761; Ralston *v.* Town of Weston, 46 W. Va. 544, 33 S. E., 326.

[57] Krause *v.* El Paso, 101 Tex. 211, 106 S. W. 121, 14 L. R. A. n. s., 582.

[58] Keller *v.* Harrison, *supra.*

[59] Koppes *v.* Koppes, 180 Iowa 1268, 163 N. W., 377; Code of Iowa, 1931, Section 12309.

[60] Fredricksen *v.* Bierent, 154 Iowa 34, 134 N. W., 432; Stone *v.* Richardson, 206 Iowa 419, 218 N. W., 332.

[61] Cheshire *v.* McCoy & Henry, 205 Iowa 474, 218 N. W., 329.

RIPARIAN BOUNDARIES

Another situation in which the surveyor's most important contribution may be in the nature of consultation rather than in the execution of surveys is that in which riparian boundaries are concerned.

A stream is a very natural boundary of lands. Even in the cases of smaller streams—and it is still more true of the larger ones—it is more convenient to cultivate or occupy separately the areas on opposite sides of the stream. As a boundary, a stream is easy to describe and comparatively unmistakable to find and recognize. It is visible—no surveyor is needed to interpret the description or trace the boundary. In the case of a small, non-navigable stream separating the lands of two owners there is little uncertainty about the boundary. The owners of the adjoining land own the bed of the stream; and their common boundary is generally stated to be the "thread of the stream," or the "center of the stream."[62] There is some disagreement among the Courts on how to find the thread or center of the stream,[63] but the weight of authority seems to be that the "thread of the stream" means the "center of the main channel" thereof.[64]

The definition of a navigable stream, for boundary purposes, differs materially in different jurisdictions. The seaboard States, following, as Colonies, the practice in England, adopted and have to a great extent retained the criterion that only streams in which the tide ebbs and flows are navigable. Obviously, this definition is not capable of sensible application in the interior of the country;[65] and most of the States other than those mentioned have adopted, substantially, the definition laid down by the Supreme Court of the United States, that, to be classed as navigable, a stream should in its natural state afford a channel for useful commerce.[66] It should be noted that if a stream has once had the character, for boundary purposes, of a navigable stream, and boundaries have thereby become determined, a subsequent declaration by legislative enactment that the stream is not navigable does not alter the boundaries.[67]

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[62] Moffett *v.* Brewer, 1 G. Greene (Iowa), 348, 358; State *v.* Livingston, 164 Iowa 31, 145 N. W., 91.

[63] Franzini *v.* Layland, 120 Wis. 72, 97 N. W., 499; Hopkins Academy *v.* Dickinson, 9 Cush. (63 Mass.), 544, 552.

[64] Franzini *v.* Layland, *supra.*

[65] Barney *v.* Keokuk, 94 U. S. 324, 338, 24 L. Ed., 224.

[66] The Montello, 11 Wall. (78 U. S.), 411; Oklahoma *v.* Texas, 258 U. S. 574, 586, 66 L. Ed., 771, 42 Sup. Ct., 406.

[67] Park Commrs. *v.* Taylor, 133 Iowa 453, 458, 108 N. W., 927; Berry *v.* Hoogendoorn, 133 Iowa 437, 440, 108 N. W., 923.

The Supreme Court of the United States has held that the extent of private ownership of lands bordering on navigable streams is for the respective States to determine. Each State may determine for itself whether it will keep the title to the bed of the stream, or whether it will relinquish this title to the private riparian owner, retaining only such an easement for public use as that retained in the case of highways where the land on which the highway is built is under private ownership. The question, then, is one of State law.[68] In Iowa, the boundary of private ownership along a navigable stream is the high-water mark.[69] A definition of high-water mark as a boundary of private ownership laid down by the United States Supreme Court and cited approvingly by the Supreme Court of Iowa is, as follows:

> "The high water mark of a navigable stream, the line delimiting its bed from its banks, is to be found by ascertaining where the presence and action of water are so usual and long continued in ordinary years as to mark upon the soil of the bed a character distinct from that of the banks in respect to vegetation and the nature of the soil."[70]

The Iowa Court also states: "What the river does not occupy long enough to wrest from vegetation, so far as to destroy its value for agriculture, is not river bed."[71]

In Iowa, however, the question of navigability of a stream is not necessarily the determining factor in the matter of the location of the riparian boundary. The Iowa Court has held that not only on navigable streams, but also on non-navigable, but meandered, streams, the boundary of the riparian owner is the high-water mark:

> "The action of the Land Department of the United States Government in meandering the stream and conveying the land bordering on such stream with reference to the meander line is conclusive that the stream was navigable in such sense that the title of the riparian owners resting on such survey extended, under the rule in this State, only to high-water mark."[72]

Hence, it would appear that the matter of riparian boundaries in Iowa is relatively clear. If during the original Government survey the stream was meandered, the boundary is the high-water mark; if it was not meandered, the boundary is the thread of the stream.

It is believed that the popular misconception that the meander line is a boundary is not generally shared by engineers, and it will not be discussed herein. Suffice it to say that a meander line can be a boundary only when there is no body of water corresponding to such meander line,[73] or when, in a very exceptional case, involving fraud or gross error on the part of the Government surveyor, the meander line is so very far from the high-water mark which it was supposed to approximate, that justice and reason demand that the meander line be substituted for the high-water mark as a private boundary.[74]

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[68] Oklahoma *v.* Texas, 258 U. S. 574, 595, 66 L. Ed., 771.

[69] Musser *v.* Hershey, 42 Iowa, 356; Barney *v.* Keokuk, 94 U. S. 324, 24 L. Ed., 224; City of Cedar Rapids *v.* Marshall, 199 Iowa 1262.

[70] Howard *v.* Ingersoll, 13 How. (54 U. S.) 381, 427, 14 L. Ed., 189.

[71] Houghton *v.* The C., D., & M. R. Co., 47 Iowa, 370, 374; Curtis *v.* Schmidt, 212 Iowa, 1279.

[72] "Park Commrs. *v.* Taylor, 133 Iowa, 453, 457–458; 108 N. W., 927; Watt *v.* Robbins. 160 Iowa 587, 601, 142 N. W., 387.

[73] Carr *v.* Moore, 119 Iowa 152, 93 N. W., 52.

Unfortunately, while a stream is a very obvious boundary, it is not always a stable one. An extended discussion of the effect on riparian boundaries of the shifting of the stream is not appropriate to the purpose of this paper. The rule is, in general, well known, that if the shifting takes place through erosion or accretion the boundary likewise shifts with the stream.[75] If the change is made by avulsion, the boundary is not thereby moved.[76] It is sometimes extremely difficult to determine whether a given case is one of erosion or avulsion. The decisions cited discuss this distinction.

Conclusion

As in many things, so in the matter of uncertain boundaries, "an ounce of prevention is worth a pound of cure." How much better it would be if surveyors and land-owners could appreciate the importance of so marking boundaries when they are first established that there will be little likelihood of their being obliterated, and of so referencing them that they can be re-established economically and precisely if that should ever be required. By the progress of the precise surveys of the United States Coast and Geodetic Survey the most reliable and satisfactory control is being made more and more widely available. While the general use of this control for the description of the boundaries of lands is probably still in the distant future, engineers should be in the lead in recognizing its utility and its value, and in urging and facilitating the adoption of this means of increasing the certainty and permanence of private boundaries.

In the meantime, and even with the use of the control mentioned, the best and most economical assurance that a boundary can be recovered in the future in its original location is the use of definite, distinctive, and enduring monuments; and the best law that can be promulgated to contribute to this end, is that which was laid down many centuries ago: "Thou shalt not remove thy neighbor's landmark, which they of old time have set."[77]

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[74] Barringer *v.* Davis, 141 Iowa 419, 427. 120 N. W., 65, and cases there cited.

[75] Philadelphia Co. *v.* Stimson, 223 U. S. 605, 624, 56 L. Ed., 570; Hohl *v.* Iowa Cent. R. Co., 162 Iowa 66, 143 N. W., 850.

[76] Nebraska *v.* Iowa, 143 U. S. 359, 36 L. Ed., 186.

[77] Deut., 19 :14.

DISCUSSION

Ray H. Skelton,[78] M. Am. Soc. C. E., (by letter).—The law of boundaries is extremely complicated and characterized by endless exceptions which Professor Holt has not stressed. However they are mentioned, and it is important to note them, because all principles laid down by the Court regarding the control of conflicting elements are rules of construction, and as such may be departed from when their rigid application defeats the true purpose of the survey, namely, the establishment of the line in its intended position.

Although the paper presents little material that is new to a competent surveyor, its publication is justified because it brings to the attention of the young engineer the "art of surveying." The field of engineering has developed so extensively in the past generation that the school curricula are crowded and, consequently, the time devoted to the fundamental subjects has been curtailed. Unless a college offers an opportunity for a student to major in surveying, it is impossible for an instructor to teach more than routine procedure. While it is not desirable that the time be taken to develop skill, it is important that the legal principles of boundaries be brought to the attention of the student, and any course that does not contain a discussion of the "art of surveying" is incomplete. Professor Holt's paper is an excellent assignment for such an exercise, and its inclusion in a course should give a student a new conception of the practice of surveying, and a foundation upon which to build if he follows the fascinating field of boundary work.

The writer regrets that Professor Holt did not include in his discussion of "A Cardinal Principle", the decision of Judge A. R. Savage, [79] of Maine,

> "The cardinal rule for the interpretation of deeds and other written instruments is the expressed intention of the parties, gathered from all parts of the instrument, giving each word its due force, and read in the light of existing conditions and circumstances. It is the intention effectively expressed, not merely surmised. This rule controls all others."

and that of Judge S. W. Sanderson,[80] of California,

> "* * * the only rule of much value—one which is frequently shadowed forth, but seldom, if ever, expressly stated in books—is to place ourselves as nearly as possible in the seats which were occupied by the parties at the time the instrument was executed; then, taking it by the four corners, read it."

These citations are admittedly vague, but they do define the foundation upon which to base an investigation for the determination of the true location. The sundry rules of construction are merely guide-posts which may or may not lead one to an understanding of the "expressed intention of parties," and are to be disregarded if they point elsewhere.

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[78] Associate Prof., Coll. of Eng., Univ. of Maryland, College Park, Md. Prof. Skelton died on October 29, 1933.

[79] Perry *v.* Buswell, 113 Maine, 399, 94 Atl. 483, 484.

[80] Walsh *v.* Hill, 38 Cal., 481, 487.

The writer once based a survey on a width of street as 60 ft, whereas the legal width at the time of the conveyance was 50 ft, because he found documentary evidence on record that the parties to the transaction had erroneously regarded the street as 60 ft wide. Consideration of this error enabled him to visualize "the seats which were occupied by the parties at the time the instrument was executed," and to lay a boundary in conformity with the deed then written and in harmony with the landmarks of to-day. Frequently, the parties sit in the wrong seats because of errors of omission. It is a too common practice to transfer properties without a survey, and frequently former descriptions are copied without regard to changes that affect them. A street is widened and in a conveyance subsequent to the widening a description that is correct with regard to the former side line of the street is erroneously copied. If one follows the rule that measurements from street lines are to be interpreted as of date of deed,[81] he will not arrive at the correct line, and it is only by giving due regard to the party's error of omission that he can establish a boundary in conformity with their intention.

Unfortunately, landmarks are transitory, and as they disappear the "seats" decay, and difficulties in interpreting the expressed intention develop. A knowledge of local history,[82] therefore, is an important part of a surveyor's equipment, and no surveyor, however competent otherwise, should attempt work in a strange community. Furthermore, in writing a deed description, it is advisable to recognize the temporal nature of all things and to base the description on the most permanent control available, so that future generations can establish beyond a shadow of a doubt the expressed intention of the parties. It is regréttable that in many cities lawyers will start a description from a theoretical point which has no existence, despite the availability of a good control.

While it is true that little use has been made of the precise surveys of the United States Coast and Geodetic Survey it is encouraging to observe that a beginning has been made. The activities of the various regional planning commissions are extending the use of exact control into the suburbs of the larger cities. With the co-operation of Mr. Edwin P. Clark, of the Brooklyn Title Guarantee and Trust Company, the engineers in charge of sub-divisions at Montauk Point, Long Island, adopted a geodetic station as the basis of their local control. A case has come before the Lower Courts involving the marketability of a title in one of these sub-divisions. The plaintiff claimed the description was such that he could not locate his property, but the Court held that inasmuch as any competent engineer could establish the lines, the title was good and marketable. The case suggests the desirability of following the technical description referred to the precise control, by a separate one which will identify the property for the layman.

The question of the location of property lines disturbed by an earthquake, is by no means academic. Charles Derleth, Jr., M. Am. Soc. C. E., in describing some effects of the San Francisco earthquake reports[83] that a property fence crossing a pipe line about 10 ft to the south of a certain rupture was offset 7 ft along the fault line and that the two pieces of the fence remained straight and parallel to their original direction.

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[81] Tebbets *v.* Estes, 52 Maine, 566; Syracuse *v.* Cook, 187 App. Div. 578, 176 N. Y. S., 396.

[82] "City Surveying for Title Examination," by Edwin P. Clark, Brooklyn Title Guarantee & Trust Co., 1921. (A local history of Brooklyn, N. Y., of interest to surveyors.)

Inasmuch as an earthquake disturbs the boundaries themselves, its possibility cannot be regarded as a weakness inherent to geodetic control alone and used as an argument against the adoption of such control. The question the writer desires to raise is: "How are boundaries adjusted after such major disturbances?" In the open country the matter is of little consequence, but where the fault crosses a city, property of high value is involved. A search of law reports has not as yet revealed a single case that has come before the Courts. Undoubtedly, the property owners recognized the catastrophe as "an act of God" and were ready to accept reasonable compromises. In the time of such disaster, relief commands the first efforts of all, and re-establishment of such essential services as water and sewerage demand the attention of the engineer, but before reconstruction can begin, disturbed street and property lines must be adjusted. There is no time for extensive situation surveys and careful consideration or the immediate recording of all adjustments. Information in regard to this secondary but, nevertheless, important problem, would be valuable to the engineer.

WILLIAM BOWIE,[84] M. AM. SOC. C. E. (by letter).—Every engineer, lawyer, and property owner should feel under obligations to Professor Holt for the help that he has given them in this paper. Much of the confusion regarding the boundaries of public and private property could be avoided by a careful co-ordination of the legal and engineering aspects of boundary location.

A phase of the boundary problem of great importance was mentioned in the concluding paragraphs of the paper where it was suggested that boundary monuments be tied into the precise surveys of the U. S. Coast and Geodetic Survey. Professor Holt's suggestion can be followed now (1933) to far greater extent than would have been possible a few years ago. More triangulation for the National fundamental net has been done during the past ten years than in the whole previous life of the nation. This increase has been due entirely to the greater need for accurate control surveys. When there was a lack of demand on the part of the people, Congress and the executive officials of the Federal Government did not feel justified in making increased appropriations for the work. Now, however, they recognize that control surveys are a public work of great value, that should be rushed to an early completion.

There now are, in the United States, approximately 40 000 miles of arcs or chains of triangles. The widths of these arcs vary. In flat, heavily wooded country the average width is from 6 to 8 miles. In mountainous areas the width of modern triangulation is approximately 20 miles. The average width for the entire country may be safely assumed to be 9 miles

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[83] *Engineering News*, Vol. LV, No. 20 (May 17, 1906), pp. 548, 550.

[84] Chf., Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

That means that 360 000 sq miles lie within the limits of the triangulation itself. There are very few places inside the arcs that are more than 10 or 12 miles from a triangulation station. The strips of land lying on each side of an arc, out to a distance of about 10 miles, are within easy reach of the triangulation stations. Therefore, there are approximately 1 000 000 sq miles, or about one-third of this country within short distances of precise triangulation stations.

The boundary situation could be improved if each State or each county would run lines of traverse between contiguous triangulation stations with enough cross-lines to form a traverse net. Traverse lines could also be extended over the 10-mile strips on either side of the arcs of triangulation. Monuments should be set at frequent intervals along these lines. The traversing of the area now controlled by triangulation could be undertaken at once and could be expanded to other areas as new triangulation is done.

There is no difficulty in running a traverse with an accuracy of at least as good as 1 part in 10 000. This accuracy is, in general, sufficient for any farm property boundary survey and for property surveys within the limits of a small settlement, such as a village or small town. The engineer with a steel or invar tape and a surveyor's transit, reading even to single minutes, can readily obtain an accuracy of 1 part in 10 000 in his traverse. This has recently been demonstrated by traverse parties in North Carolina, working under the direction of the State Highway and Public Works Commission.

The Coast and Geodetic Survey has recently taken a long step forward in adopting single-plane co-ordinate systems for the States of North Carolina and New Jersey and for the area of Long Island. In New Jersey, the scale factors have a maximum value of 1 part in 30 000, except for a very small part of the area in the southwestern part of the State. On Long Island, the maximum scale factor is somewhat smaller than 1 part in 100 000. The greater expanse of North Carolina results in a somewhat larger factor for that State, but one that is quite satisfactory to the State Highway and Public Works Commission which co-operated in the adoption of the system.

There are many engineers who hesitate to use triangulation and traverse data which are based on spherical co-ordinates, that is, on latitudes, longitudes, and azimuths (or true bearings), but who find no difficulty whatever in using the results of such triangulation when the positions are computed on a rectangular system. Rectangular systems will undoubtedly be adopted for other States but, in many States, it will be necessary to have more than one system because of the great width involved. A State with a width of less than 158 miles in the east-west or north-south direction can have a single co-ordinate system with scale factors not greater than 1 part in 10 000.

The tying in of property surveys, whether public or private, to the triangulation system of the country safeguards the rights of the property owners. There will be little or no difficulty in recovering, at any time in the future, a property line connected to control surveys even if all the monuments that were originally set, have been destroyed. Markers may be destroyed, but co-ordinates are permanent as long as a few stations of the triangulation net of the country remain in place. The mortality of triangulation station marks is exceedingly small, and the property owner may rest assured that, when his lines are tied into the National net, he and his heirs will always be able to recover their lines.

In the State of New York the referencing of property surveys to the National triangulation system has recently been given legal approval through a decision of the State Supreme Court.

R. ROBINSON ROWE,[85] M. AM. SOC. C. E. (by letter).—The practice of surveying has enjoyed unusual prominence in recent publications of the Society, and there is need for much more. The comparison[86] by Mr. M. L. Greeley of the high standards and standing of the profession in Canada, with the Kentucky decision of 1920 that surveying was a trade, is not a great exaggeration, unfortunately, of the actual status. Professor Holt's timely paper suggests not only a contributing cause, but also some of the remedies. Since the Profession of Civil Engineering includes surveyors, inevitably time will level them—either to the rank of professions or to the file of the crafts. Hence, the interest of all civil engineers, even though many do not practise surveying.

The great demand for resurveys of residential lots during periods of active construction has presented the opportunity for unqualified men to establish themselves as surveyors. Experience as instrumentmen under competent party chiefs may have given them sufficient aptitude in the use of the instrumental equipment, but it did not often give them proficiency in the use of "legal equipment." Their surveys were ordinarily dependable, if acceptable monuments were evident and the original survey well done. Usually, they depended upon offset lines or witness corners of city or county surveyors and if, occasionally, they were too trusting in the infallibility of such officers, at least they enjoyed a good alibi. When such evidence failed, still the survey was made; no problem was too difficult for a solution. North was north, and 100 ft was 100 ft, in the description and on the ground. These surveyors knew no law except that of single proportion and no logic except "confidence begets confidence."

Their overhead expense was nominal, based on education gained on the job, equipment rented or borrowed, residence telephone with a business classification, and assistants paid by the hour. When the depression came, they reduced prices and solicited business to keep busy. Since careful surveyors would not compete with such tactics, the business of the latter was soon limited to a small discriminating clientele, surveys of litigated property, etc.

Conflicts between such careless surveys, price-cutting tactics, solicitation, and the type of man who now claims to be a surveyor, have made a trade of lot resurveying and kindred work, yet most of these men tried to do the right thing. Ignorance of the law, rather than inaccurate field work, caused the conflicts; ignorance of ethics rather than lack of conscience, led to price cutting and solicitation.

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[86] *Civil Engineering*, September 1933, p. 510.

The remedies consist of eliminating the unfit and educating the fit in the legal equipment and ethics of surveying; compiling for all surveyors, the essence of the law governing surveys, not only for Minnesota and Iowa, but for all States; and obtaining such benefits as laws may make available to assure an increasing respect for the profession. The following suggestions are offered as initial steps toward definite remedies:

First.—A code or standard of practice for surveys and resurveys should be compiled and published, including ethics, common law, and minimum requirements for accuracy and monumentation. The code might determine the availability of one surveyor's notes to another, or require the making of a public record of the restoration or perpetuation of monuments, or even require perpetuation or witnessing of monuments in danger of obliteration.

Second.—Digests of common law, statutes, and decisions should be compiled, showing for each State, if different, the statute of limitations, leading or ruling decisions on adverse possession, acquiescence, riparian boundaries, proportion, and all other matters which have led to litigation with admitted facts, and, also, liability of surveyors for damages resulting from mistakes or trespass.

Third.—An investigation should be made of the possibility of legalizing arbitration as a means of determining boundaries, where the decision must be based on facts rather than on law. Such procedure would save money, time, and tempers.

Fourth.—The desirability of assuring the accuracy of surveys should be ascertained by an investigation of legal phases, probable costs, possible demand, and ethics involved. Buyers of real property usually insist upon title guaranty or insurance, but presume that possession is proof of agreement between deed description and lines on the ground as indicated by the seller. Further insurance might be demanded if available.

The cost would be that of a careful survey, premium on the risk, and a nominal charge for plat and certificate. The writer can conceive some advantages to this, namely, that insurance cost would be determined by value of property, its proposed use, and particularly by the development up to the property line, as well as the ability of the engineer (based at first upon reputation and later on merit rating). Hence, the careful scrutiny of the insurance companies would eliminate irregular surveyors from this class of work by setting high premiums. On involved surveys, the insurer might require consulting surveyors or attorneys to review the work. There would be a tendency of owners ordering uninsured surveys to prefer the services of surveyors qualified to make insured surveys.

The important thing is to raise the standards and standing of the surveyor. Professor Holt is well within his rights in stressing many apparently simple principles that should govern the conduct of resurveys. Until these and many other principles of law and ethics are recognized by surveyors, the profession will remain a trade.

WALTER H. DUNLAP,[87] ASSOC. M. AM. SOC. C. E. (by letter.)—It is difficult to add to, or subtract from, Professor Holt's excellent exposition of the legal principles pertaining to boundaries. The discussion herewith aims to present an historical background to account for, and to illustrate, the principles set forth by the author.

Doctrine of Adverse Possession.—In the early days of Virginian history, Charles II of England freely made extensive grants of wild land in America to favorites and as rewards for public services, regardless of the fact that such land was already occupied by others to whom he had previously granted it. In 1684, the rapacious and unprincipled Lord Culpepper, whose chief interest in Virginia was exploitation for his own profit rather than benefit to the growing colony, was confirmed by the Crown in his title to the Northern Neck of Virginia, being that slender peninsula lying between the Potomac and Rappahannock Rivers.

At that time little was known of the country west of the fall line (Fredericksburg), but in 1716 the public spirited Spottswood led his Knights of the Golden Horseshoe across the Blue Ridge Mountains and discovered the Shenandoah Valley, whereupon Lord Fairfax, grandson of Culpepper and heir to the Northern Neck, claimed all the land between the two rivers all the way back to the source of each, which was quite an indefinite boundary. Fairfax later established himself in the Valley at "Greenway Court," near Winchester, and proceeded to dispose of his lands in parcels called "quit claims." His surveyor was George Washington.

In the meantime, others had settled on the land. About the time (1607) of the first permanent colony in America at Jamestown, James I also planted in the northern part of Ireland a colony of picked men and women from Scotland, and by the beginning of the Eighteenth Century the colony had become so strong and its textile industries so prosperous as to engender violent jealousy on the part of English manufacturers. Religious intolerance and economic persecution drove many of these Presbyterian Ulstermen to emigrate to America where they pierced through the seaboard settlements and became a kind of buffer State against the Indians. They stopped in Pennsylvania long enough to learn the wisdom of purchasing rather than grabbing the land from the Indians; but this policy was not so easy to follow because the Valley of Virginia was the common hunting ground, and, consequently, the battleground of several warlike tribes.

Such were the Scotch-Irish settlers whom Lord Fairfax found on "his" land. Theodore Roosevelt calls them the first real Americans, because long before the Cavalier settlements in Tidewater Virginia had ceased to give full loyalty to the Crown, these rugged settlers, exposed to Indian warfare from which they were entirely unprotected by the Crown, recognized no earthly sovereign. When Fairfax undertook to claim "their" lands, stubborn disputes followed, which were carried on in the Courts for a full hundred years.

The present doctrine of Adverse Possession and Squatter's Rights is undoubtedly based in considerable measure on the law as developed in those long drawn-out legal battles between the Cavalier proprietors and the Puritan squatters. Thomas Jefferson has sometimes been called the father of American independence and the Shenandoah Valley its cradle.

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Retracing Old Boundaries.—Land titles in Eastern Kentucky present some difficult and interesting problems in the application of the principles recited by Professor Holt.

During the last decade of the Nineteenth Century and the first decade of the present century, coal land organizers in Eastern Kentucky were very active in the business of acquiring from the small farm owners, titles in fee simple or for mineral rights to as many adjacent parcels as possible in the effort to block up areas sufficiently large to justify large-scale mining operations. One of the most successful of these organizers was the late John C. C. Mayo, of Paintsville, Ky.

In 1906, Mr. Mayo outlined an area of about 30 000 acres mostly in Johnson County, put lawyers to work abstracting titles, and organized a corps of surveyors and draftsmen, of which latter group the writer was a member. Transit traverses were carefully run up the main streams and through the gaps to form great loops, which, in turn, were divided into smaller loops by tie lines. Latitudes and departures were computed for closure and for co-ordinates, which latter were used in mapping. The metes and bounds of every separately owned tract were then run out by land surveyors with compass and tape, and tied into the transit traverse for control.

The survey was platted on sheets of mounted paper 5 ft wide by about 12 to 15 ft long, on a scale of 400 ft to an inch, and areas were taken with a planimeter. At any time an owner was inclined to sell, and a price per acre was agreed upon, the deed containing the surveyor's description could be written and the deal closed within an hour.

In many cases it was found necessary to have the surveyors run out not only the lines called for in the deed, making notes of fences, clearings, years of occupancy as claimed, etc., but also the underlying patents as well. These patents merit a short description.

About 1775 white men began to settle south of the Ohio River, and for the next twenty-five years, or more, the authorities of Colonial Virginia and the new State of Virginia granted to various persons letters patent to large, indefinitely described, unsurveyed or partly surveyed tracts of land in the eastern mountainous part of what became in 1792 the State of Kentucky. The new State likewise granted patents for such tracts, neither knowing nor greatly caring that much of the land was already covered several layers deep by such grants, or that *bona fide* settlers actually occupied it to an increasing extent.

About 1800, these mountain settlers began to take out patents covering the narrow meandering strips of bottom-land they had cleared. These grants, known as "blacksnake surveys" from their shape, were followed some years later by larger surveys made for the purpose of including the mountain land to or beyond the top of the ridge.

The practise of the old surveyors in making these surveys is well authenticated. They actually measured the metes and bounds along one side of the clearing, using standing trees as corners, which they marked and described. At the beginning tree and at the last tree of this measured traverse they usually sighted the compass roughly in the direction desired, recorded the bearing, and went back to the house. In the "office" they platted this base line and drew other lines which they judged would enclose the areas desired. In writing the description they protracted and scaled the lines not measured in the field, and described each of the unsurveyed corners as being marked by a stake. The description always closed with the words "containing blank acres more or less."

When these old descriptions are re-run in later years, one or more of the marked trees of the base line can often be found, but the distances almost invariably fall appreciably short of those recorded by the early surveyor. The problem of how to adjust the old stake calls to the corrected base then arises.

A basic principle of land surveying is that the description must be reasonably construed. It is easy enough to accept the more accurate measurements of the recent survey as correctly describing the line from one known marked tree to another, and to swing the entire survey in azimuth enough to offset the change in magnetic declination, but it is more difficult to set up a solution that will adjust the stake calls more reasonably than any other that may be proposed by an opposing claimant.

The usual procedure is to plat the original description and—if it closes reasonably well and if the acreage approximates that intended to be enclosed by the original proprietor—to claim that the stake calls were not fictitious, but real lines, surveyed on the drawing-board rather than on the ground, and fitted more or less truly to the area in the minds of the surveyor and his employer. This reasoning and the principle that lines may be run backward as well as forward permit the entire error to be thrown into the first stake call; or, if other circumstances indicate that it is more reasonable to throw it into the line from the last stake to the beginning corner, that can be done.

It is a difficult problem at best; it is not an academic one at all, because such surveys are very, very numerous, and the coal underlying the surface is valuable. In tracing such descriptions the surveyor must proceed with confidence because one or more of the adjoining land-owners usually accompany the survey party, and the men of the Southern Appalachians are notoriously contentious for their rights. Land disputes constitute the major cause at the root of most of the family feuds among them.

The old surveyor usually measured directly along the slope rather than on the horizontal, and was not too careful in chaining to keep to the straight line connecting the points. As a consequence there was much overlapping of surveys from opposite sides of a ridge. Some surveyors kept the chain tally by tying knots in the leather thong at the end of the chain, and if one or more knots were lost in dragging the chain through the bushes, error resulted.

Apologies must be made to the old surveyors whose mistakes are emphasized herein for the purpose of describing the problems presented to the land surveyor of the present. Most of their work was as good as could be expected with the instruments and help at hand, done as it was at a time when a good jack-knife sufficed to purchase a number of acres of land that covered coal deposits which are now worth thousands of dollars.

Chester Mueller,[88] Assoc. M. Am. Soc. C. E. (by letter).—A seldom mentioned but vitally important phase of surveying is described in this paper, namely, its legal aspects. In a limited space Professor Holt has struck to the heart of each of the major legal problems of a surveyor with clear, concise explanation that permits of no criticism and leaves nothing to be added except some general comments.

That more time should be spent, in engineering courses, on those fundamentals of law that are closely interwoven with the technical professions is conceded by most engineers. The writer has found that a knowledge of law in engineering work is a valuable asset in many respects.

Professor Holt lays stress on the judgment that a surveyor should exercise in order to do his client (and adjoining land-owners) justice. To any one who has served his apprenticeship under a surveyor of the "old school," the meaning is plain. Frequently, a surveyor capable of running lines with precision fails to delineate boundaries correctly, due to such lack of judgment. When such a man has seen a more experienced surveyor define those same boundaries he feels a strange sense of uncertainty in boundary work because so much depends on elements other than a correctly graduated tape and accurately adjusted transit. Surveyors engaged in re-running old lines do well to proceed with caution before insisting to their clients that their work and not the preceding work should be given credence.

When physical signs on the ground run counter to even the most precise survey it ill behooves a surveyor to dismiss them from consideration without first subjecting them to a careful analysis. Litigation and disputes between former friendly neighbors is often due, as Professor Holt observes, to the inexperience of a surveyor. When differences appear between adjoining surveys, that cannot be dissolved by the surveyor last on the ground, it is the general practice for him to consult his predecessors and by each surveyor contributing his knowledge of the facts in the case a resulting line is determined that renders justice to each of the abutting land-owners. Perhaps, in a measure, the success of such informal arbitration in New Jersey accounts for the relatively few reported boundary cases in recent years. This is all the more unusual because as one of the thirteen original States, New Jersey's land grants extend back to 1664 and were never subject to a systematic geodetic control. In the many years since those early days more refined methods of surveying were naturally grafted on to many of the old lines established by descriptions drawn in Colonial days, with resulting complications.

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Two cases, one as recent as 1933, hearken back to pre-revolutionary days and furnish typical examples of how the new lines are interpreted in view of the old. The case of Warner *et al v.* Smith *et al* (164, Atl. 282, 112 N. J. E., 152) required a decision based on a grant made on February 17, 1742, by the Proprietors of West Jersey which covered three islands and which described them by name and acreage only, and on a grant made by the same Proprietors in 1776 of an island described by metes and bounds. The established rule of law, which holds metes and bounds to be controlling over quantity, was followed in this case, but the application of that rule appeared somewhat difficult due to the fact that to-day only three islands remain, whereas the two grants combined called for four. Furthermore, one of the three islands first granted was described as Pork Island and called for 17 acres, whereas the second grant describing Beach Island by metes and bounds and calling for 244 acres, was claimed to apply to the island which to-day is called "Pork Island."

The claimants under the 1742 grant were successful in showing that the acreage reported under that grant was unreliable, but were unable, however, to demonstrate that an island granted as 17 acres in 1742, was the same one granted as 244 in 1776 (accurately surveyed as 312 in 1841); nor were they able to disprove the defendants' evidence that Beach Island first assumed the name of "Pork Island" by popular usage many years after 1776. The case is principally of historical interest to surveyors, but also shows how the fundamental boundary issues may be obscured as a case develops.

The other case decided in 1889 was that of Scott *v.* Yard (18 Atl. 359, 46 N. J. E., 79) and brings out an interesting situation in which a claimant attempted to "take up," legally, what he insisted to be unappropriated land. Scott, the plaintiff, acquired on August 26, 1880, three lots of land fronting on Ocean Avenue, in Spring Lake, N. J., each described as being 50 ft in frontage by 150 ft in depth. His land lay on the west side of Ocean Avenue, the Atlantic Ocean bordering on the east side of the Avenue. Running back Scott's title, two tracts are involved. The west part of his lots were in the Forman tract which was surveyed and allotted to Forman in July, 1746, and the east portion of his lots were in the Brinley tract which was surveyed and allotted on October 1, 1860. The defendant claimed title to a strip crossing Scott's lots and lying between the two tracts mentioned, by reason of an including survey (approved by the Surveyor-General on October 19, 1800) the nature of which was to embrace land previously surveyed and appropriated by other persons and to "pick up," in the language of the Court, any land unappropriated.

The Court held that the Brinley survey of 1860 contemplated including all the land between the ocean and the east line of the Forman survey, and only by mistake would a surveyor fail to include all the land possible for his client. In this connection, it ruled that surveyors of the past did that which an ordinarily skilful, proper discharge of their duty to their principals required them to do.

Interestingly enough, the Court also brought out the fact that the rules of the Board of Proprietors prohibited its surveyors from fixing locations and lengths of lines by monuments and directed them not to mention monuments in the description of lands they surveyed, except at the beginning point.

An examination of the reported cases which discuss the surveyor as a witness, and what may be evidential in determining land boundaries, will throw considerable light on the opinion in which the Courts hold the functions of a surveyor. The only boundary case reported in New Jersey in 1932[89] contributes something in this respect. The case came to the Supreme Court on the defendant's argument that the verdict rendered in the Circuit Court was against the weight of evidence and that, therefore, a new trial should be granted. In its opinion the Supreme Court stated that "it suffices to say that the testimony of plaintiff's surveyor, a man of many years experience, was subject to credence by the jury and was not overcome by any evidence of a convincing character offered by defendants," and dismissed the defendant's rule.

While informal arbitration and existing statutes are deemed of value, the surveyors of New Jersey have recently been urging some more suitable means, fully countenanced by the law, to adjust boundary disputes and irregularities and to prevent their occurrence. The Legislative Committee of the New Jersey Association of Professional Engineers and Land Surveyors has been giving considerable study to the problem of clarifying and strengthening the law in this respect. Preparation of descriptions by competent surveyors following uniform regulations, filing and recording of survey plans, and properly fixing property boundaries by monuments, are typical of the subjects embraced in the study.

Outstanding in its treatment of land boundary matters is the State of Massachusetts which has a Land Court, that among other duties has exclusive original jurisdiction of petitions to determine the boundaries of lands. The act (Chapter 185 of the General Laws of Massachusetts) which established the Court also provides for the registration of the title to land.[90] The act creating the Court, and the Court's published instructions, furnish excellent material for a basis upon which to redraft legislation in other States.

Searching for the "footsteps of the surveyor," of which Professor Holt writes, is essential to the proper exercise of surveying judgment. When working in the field with his father, the late Carl Mueller, who spent a lifetime in the profession, it was a constant source of revelation to the writer how unerringly he sought out and found such "evidence," with a method appropriate to the locality. The result of discriminating evaluation of such evidence was that frequently in driving a corner stake it would hit on a buried remainder of its early predecessor. As aptly expressed by Professor Holt, the cardinal principle is not to correct an established boundary line, but to reproduce it on the ground, and perhaps no better method can be followed than to attempt to retrace, both mentally and physically, the first surveyor's footsteps.

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[89] Girard *v.* Stager, 156 Atl. 14, 9 N. J. Misc., 871.

[90] *Civil Engineering,* August, 1932, p. 506.

Admitting that surveyors should be familiar with the legal principles encountered in their daily work, it is not amiss to point out that many, if not most, lawyers are totally uninformed as to the nature of the work of a surveyor. They, too, believe that a surveyor following a description (written loosely, or otherwise, by a layman or by a lawyer, more often than a surveyor) should establish the boundaries without effort and with certainty. They do not, as a rule, know what temperature changes do to tapes, why compass bearings vary, and how the many other factors may introduce differences in the surveys of the same boundaries at different times. Above all, they do not realize that the value of land a century or more ago did not warrant the accuracy called for to-day and, therefore, they do not interpret the results accordingly. If surveyors in those cases of necessary contact with lawyers would always keep in mind that they must first "educate" the lawyer in fundamentals before they can hope to have him understand the specific case at hand, they would more readily establish mutual respect and understanding.

As an illustration of the common belief in "measure" as absolute, there is the interesting argument advanced by counsel in a case[91] decided in 1881, a time when most principles of law concerning boundaries in the United States were already well established. The case involved the survey of a boundary line that differed from another survey by 0.29 in. for each 50 ft of length, attributed to the difference in length of the respective 50-ft chains used. An iron monument was found at the disputed corner, but nevertheless the attorney for the defendant argued that since the United States had fixed a standard of length by law, that standard and no other controlled the distance measured along the boundary. In answer, the Court held that a distance gave way to a monument, that a standard unit of measure was not relevant to the issue, and that in its opinion no statute could constitutionally cause boundaries to be determined by measure only.

While the principles of law as applied to land boundaries may be clear, it is their application that presents the most difficulty. For example, a lot was sold by description, as beginning at a marble monument set in the side line of a street. Actually, the monument was a few feet removed from the side line of the street. Analyzing the wording of the description to discover the apparent intent, the Court held that the street itself was a monument and as a "natural" one took precedence over the "artificial" monument and that, therefore, the beginning point of the course lay in the side line of the street—a decision that really discredits the original survey, but actually carries out the intent of the parties.

Another instance where intent was carefully scrutinized and where a distance gave way to a physical boundary arose in a case[92] decided in 1880. The deed conveyed a house and the lot on which it stood, among a row of houses, and the description in reference to the depth of the lot read, in part,

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[91] Kalbfleisch *v.* Standard Oil Co., 43 N. J. L., 259.

[92] Smith *v.* Negbauer, 42 N. J. L., 305.

"thence easterly at right angles to Grove Street 80 feet or a fraction more or less * * *". The grantor only owned 65 ft in depth, a fence standing at this distance back from, and parallel to, Grove Street. The Lower Court gave the plaintiff who was the purchaser, the value of an additional 15 ft, but the Supreme Court resting on the admission of the plaintiff that he had viewed the property prior to purchase and had believed the fence to be on the line, reversed the decision. The perusal of such cases as this one serves to enlighten a surveyor considerably on the reasoning supporting the judicial opinions which have established the legal principles involved.

The beacon for New Jersey jurists, embodying the principles of the law concerning boundaries, is perhaps best expressed in the following opinion of Chief Justice Green in a case[93] decided in 1859:

> "The rule is well settled that boundaries may be proved by every kind of evidence that is admissible to establish any other fact. Actual occupation, ancient reputation, the admission of the party in possession against his interest, ancient maps and drafts, marked trees, the line of adjoining surveys, monuments erected at or soon after the date of the grant of adjoining surveys, are all admissible for this purpose, and are constantly resorted to fix the boundaries though it conflicts with the courses and distances called for in the deed. The well settled principle is, that practical location is evidence of a mistake in the description."

Chief Justice Green points out that the practice as well as the reason upon which it rests, is stated admirably by Mr. Justice Washington in Connecticut *v.* Pennsylvania (1 Pet. C. C. 511), as follows:

> "No gentleman of the profession who is at all conversant with land trials, can be ignorant that the courses and distances laid down in a survey, especially if it be ancient, are never, in practice, considered conclusive, but that, on the contrary, they are liable to be materially changed by oral proof or other evidence tending to prove that the documentary lines are not those actually run. How often we have known reputed boundaries proved by the testimony of aged witnesses and by such evidence established in opposition to the most precise calls of an ancient patent. Such evidence has been constantly received, and distances have been lengthened or shortened without the slightest regard to the calls of the patent. The reason is obvious. It is not the lines reported, but the lines actually run by the surveyor which vest in the patentee a title to the area included within those lines. The survey returned on the patent is the evidence of the former, natural marks or reputation is in almost all cases the evidence of the latter. The mistakes committed by surveyors and chainbearers, more particularly in an unsettled country and wilderness, have been so common, and are so generally acknowledged as to have given rise to a principle of law as well settled as any which enters into the land titles of this country, which is, that when the mistake is shown by satisfactory proof, courts of law, as well as of equity, have looked beyond the patent to correct it."

Professor Holt's paper serves to point the way to more careful boundary surveys and should encourage those of the profession engaged in such work to a further study of the two reference texts he cites.

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[93] Opdyke *v.* Stephens, 28 N. J. L., 83.

Clarence T. Johnston,[94] M. Am. Soc. C. E. (by letter).—The author has brought together in compact form many details of the law relating to boundaries which concern the surveyor and influence his professional activities. Volumes pertaining to the same subject have been compiled by authorities in the law. Several works have been written by men who represent both legal and scientific opinion. Those who specialize in boundary surveys have long felt the need of a digest of the essential element of the law. To select the important features from the accumulated mass, represents a task which, to resort to paraphrase, makes surveyors lose the good they might otherwise win by fearing to attempt.

The practical experience and special interests of surveyors influence their judgments regarding the relative importance of specific provisions of the law. Many surveyors chafe under the limitations that the law seems to prescribe. If the surveyors who made the basic surveys had been adequately supported at the time such surveys were made, much of the existing law would now have but little weight or meaning. Specifications defining enduring monuments are not to be found in the common law. The surveyor is often ready to condemn much of this law as it relates to his work because it ignores science and because it does not refer to standard monuments, limiting closing errors, and other items of major importance to him.

As the essential features of the law are brought together and discussed by those who have conflicting opinions, a foundation should be laid for a common understanding among surveyors. Professor Holt's paper is sufficiently comprehensive to form a basis for a national discussion of the law relating to boundary surveys. When the discussion is completed and final conclusions are drawn, the essential features of the law should be outlined and recorded in terms that are easily understood. This may lead to the enactment of statutes which reflect the best of modern practice.

The law relating to boundary surveys has not improved to keep pace with the growth of scientific theory, reformed practice, and better equipment. Science has long since prepared the way for the introduction of a new era in boundary surveying. Surveying and land tenure began to be recognized when early peoples departed from nomadic customs and engaged in occupations that involved settlement and the cultivation of the soil. Science was in its infancy when lands were parceled out to individuals and families. The rules governing the occupation and use of lands became tribal traditions. They were finally incorporated in the "unwritten law" and modified to meet the necessities of many early peoples. The law continued to grow as life became more complex. Science had but little application to boundary surveys until long after the development of comprehensive codes relating to land tenure. The power to make final decisions was then, as now, entrusted to those trained in the law. It is not a matter of accident that there are volumes of legal discussion relating to the boundaries of real property and a scanty literature dealing with the applications of science to the same subject. The process of evolution is extremely slow as traditional doctrine and opinion undergo modification to meet, in a small way, the requirements of a changing society. As specifications based on science replace legal authority, the law relating to boundaries must be simplified.

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Surveyors are obliged to familiarize themselves with the law as it exists because the power to establish boundaries rests with the Legal Profession and the Court. There are unmistakable signs of agreement among surveyors as to the logical first steps they should take to hasten reform. A review of boundary surveying in the United States discloses an almost uniform carelessness regarding monuments. Distinctive, enduring monuments marking the intersection of boundaries should be the objectives of every boundary survey. While the quality of work performed in connection with many original surveys was not high, errors in linear and angular measurements and discrepancies in the official records may well be overlooked. Carelessness relative to monuments cannot be condoned. The cost of distinctive, enduring monuments is trifling compared with the total cost of public or private surveys. Failure to set such monuments evidences a lack of appreciation of the first objective of a boundary survey.

It is recommended, therefore, that standard monuments, such as are used by the General Land Office, the United States Coast and Geodetic Survey, and other departments of the Federal Government, be set to mark the intersections of real property boundaries in all future surveys. Distinctive, enduring monuments will be seen and respected. Freeholders, and the public generally, will become boundary conscious. An appreciation of good monuments should soon lead to triangulation control, local systems of co-ordinates, and complete records of all surveys in appropriate public offices.

The surveyor who is retained by one or more freeholders to set monuments and thus mark real property boundaries, has responsibilities which are often overlooked. In the first place, he should have the technical qualifications that are recognized by the leaders of his profession. He should be familiar with the law as it is outlined by Professor Holt and by those who make positive contributions to the discussion of his paper. He should understand that his work should be performed in such manner as to merit the confidence of his clients and the respect of other interested parties. Whether the locations of monuments are determined by agreement or by order of the Court, he should urge the setting of standard monuments. He should understand that all boundary surveys are matters of local public concern, and he should leave a plain and complete record of his work in some designated public office. A surveyor who regards the notes and computations to boundary surveys as private property does not exhibit a professional attitude toward his work. In the long run his own practice suffers as his competitors pursue the same selfish and short-sighted policy.

In his concluding sentences Professor Holt refers to the value of "definite, distinctive, and enduring monuments." If all real property boundaries were to be marked by such monuments, the public would be afforded the basic protection that surveys should afford. The setting of such monuments would introduce a new era in boundary surveying.

A. F. Harley,[95] M. Am. Soc. C. E. (by letter).—The analytical nature of this paper is its principal charm, and its publication becomes at once a graceful introduction to the works of Clark and Skelton.

At the very outset of the author's remarks one quickly discerns the difference between a lost and an obscured corner. In ninety-nine cases out of one hundred, "lost" corners are merely obliterated corners, and their finding is as interesting as a jig-saw puzzle.

Before 1912, however, the writer did not think so, and was prepared to swear that each obliterated corner in cut-over pine woods was in every sense of the word a lost corner, since all the old trees, "witnessed," "blazed and hacked," or merely good round timber had been cut and carried to the nearest saw-mill. Then he happened to read a paper[96] by N. B. Sweitzer, Assoc. M. Am. Soc. C. E., describing a method of finding the original pits surrounding a section corner mound, which had been filled in, and leveled down by the wind, in the flat, dry, desert soil of Nebraska, and concluded, that in comparison the task of finding obscured corners in the cut-over pine woods of Florida was comparatively an easy one. This it not only proved to be, but it held a Judge of the United States Court in close attention, until the method pursued had demonstrated its unerring precision.

The writer's method is as follows: Approximately, at an obscured section corner a stake is driven, the transit set up, the vernier set on zero, the telescope pointed toward the north, and azimuth angles and stadia distances are read and recorded to every stump within a radius of 200 ft, or more, from the instrument.

The rodman is previously instructed to hold his rod on every stump, or what may remain of the tap-root of a tree, or at the center of surface lateral roots, running out from where an old stump had been, and to cut a chip from each, before going to the next stump. This done, he is again told to walk backward and forward over the ground and especially through high grass and briars to look for other stumps he may have overlooked. These angles and distances are then platted in the office, and each stump is shown by a black dot; the drawing is finished with a north point.

The witness trees of the particular section corner, as marked by a United States Deputy Surveyor and recorded in his field notes under instruction from Robert Butler, Surveyor-General of the then Territory of Florida, in 1848 and 1849, are then platted on a piece of tracing paper, and the four or less witness trees shown by small circles; and this drawing is also finished with a north point.

Then commences the jig-saw puzzle, until a spontaneous exclamation from the draftsman of "I have it" indicates that while keeping the north point on the tracing paper parallel with the north point on the drawing beneath it, and moving the tracing paper up and down and from side to side, four black dots suddenly appeared within the four circles.

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[95] Cons. Engr., Jacksonville, Fla.

[96] *Transactions*, Am. Soc. C. E., Vol. LXXV (December, 1912), p. 393.

Fig. 1 is an example from the public records of Duval County, Florida, showing this indisputable evidence of the true position at which 8 by 8-in. concrete monuments have been placed to mark for all time certain heretofore obscured section corners and quarter-section posts in Duval County. The dots represent the trees or stumps of trees; the dots enclosed in a circle are the trees or stumps of trees the bearing and distance of which conform to the bearing and distance of the witness trees as recorded in the field notes of the U. S. Deputy Surveyor when surveying public lands, and now preserved in the State Capitol at Tallahassee, Fla.

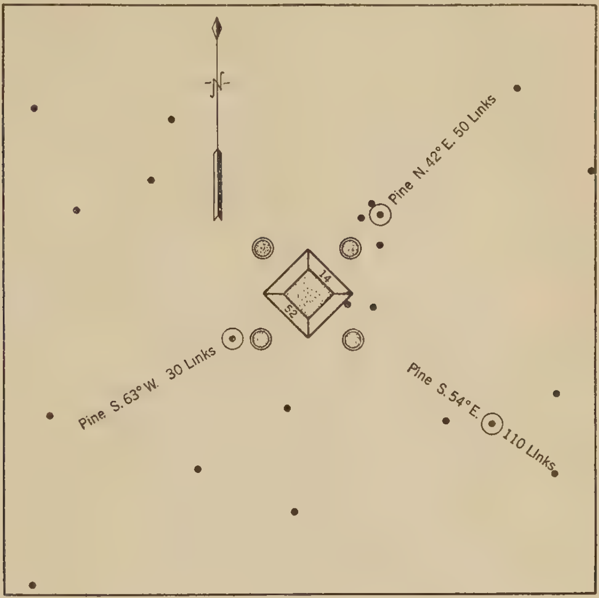

Fig. 1.

Fig. 1 also shows the 8 by 8-in. concrete monuments (drawn to a larger scale), turned so that the section numbers cast on the side of the concrete monuments may face the sections to which the numbers refer. It also shows four shaded circles which represent four small circular concrete monuments, buried entirely below the surface. These witness monuments would enable a section corner monument that had been moved, to be replaced in its original position, or they would enable a surveyor to substitute a new section corner monument for one that had been broken.

Of a truth, the men engaged in this work had found the footsteps of the surveyor; and more than this, they actually knew, after the final transit lines had been run from monument to monument, the average error in measurements the original surveyor had made under his contract with the Government on a per mile basis of compensation.

Cost? Oh yes, it costs, but Uncle Sam did not learn from his predecessor, who was equally generous in giving homestead lands away to the early settler

In California, the receiver of a gift of land from Spain was heavily fined if within a reasonable time thereafter, properly laid-up stone monuments did not mark the boundaries of the gift.

In Florida, the public lands were reasonably well surveyed. Henry Washington was one of Colonel Butler's Deputy Surveyors under contract, and one of his best. In one of the Spanish Grants, Henry Washington ran 75 450 ft, or 14.3 miles, on 61 different courses, and his notes were returned with a closing error of 32 links east and west, and 35 links north and south.

Being somewhat skeptical of the result and yet having occasion to use it for the information it supplied, the writer re-calculated the entire traverse, carrying through the plus and minus quantities from course to course and platting the results on a 400-ft scale, which brought the end of the last course 72 ft from the place of beginning, or slightly less than 1 in 1 000.

One might well ask how it was done in Florida one hundred years ago (1834). Fudged? The writer thinks not. Washington had, of course, a compass with a nonius and two 2 pole chains, subdivided into 50 equal links, as required in the instructions of Colonel Butler to be supplied by each Deputy, and what more could a tradesman need?

Call surveying a trade if you will, but the occupation demands considerable "horse sense." A surveyor should know the direction he is facing at all times just as sailors do. He should know the names and the probable life of different trees. If the tree is dead its species can be determined from the grain and scent of the wood. The surveyor should be able to recognize new growth over an old scar, and should know how to remove such new growth and expose the old scribing. He should know by rote the instructions of the Surveyor General appointed by Congress to conduct the survey of an acquired territory, which were issued to the Deputy Surveyors in the field, thirty years before the first Manual published by the General Land Office ever left Washington, D. C., and he should also know the *modus operandi* of that Surveyor General in carrying out the orders of Congress. Knowing these things he will have learned the first three letters of the alphabet of the surveyor's trade.

LYNN PERRY,[97] M. AM. SOC. C. E. (by letter).—The subject-matter of this paper is timely, and the author is to be complimented on the thorough and interesting manner in which he has handled a subject so difficult to present clearly. The paper is well authenticated and well annotated. Although many of the citations appear to be somewhat local, the legal principles laid down by the Courts in the excerpts cited are general, and the inexperienced surveyor will do well to follow them.

Many engineers, at some period during their professional career, become involved in boundary disputes and property lines. To some, this duty comes at a time when they are least adequately prepared, legally, to render the decision that is expected of them. The attention of the writer was drawn to the logical legal evaluation of the "calls" in the description of a property, early in his career. He was surveying a tract of land in Maryland, the northern boundary of which was described as "thence, due east" a certain distance "along the Mason and Dixon line dividing Maryland from Delaware." The Mason and Dixon line at this point was monumented in 1760, after about nine years of surveying, injunctions, agreements, Court decisions, etc. More recent researches have disclosed that Mason and Dixon probably did not inspect the field work of that part of the line themselves; they merely approved the manner in which it had been done and authorized the line to be monumented. Much to the surprise of the immature surveyor, it was found that the monuments were not in an easterly line (in fact, they were not in line at all); they were not at 1-mile intervals, except approximately; and the east and west ends of the client's holdings in no way corresponded with the distance mentioned in the description. Obviously, the property held under a Maryland patent for more than 140 years was in no way a part of Delaware, and the northern boundary was made, not due east, but a meandered line connecting the existing monuments.

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[97] Asst. Prof., Hydr. and San. Eng., Lafayette Coll., Easton, Pa.

An interesting feature of this survey is that one monument was missing and was found being used as a door step by a near-by tenant farmer. The man had no idea of its history, as it had been used as a door step prior to his occupancy of the property. He was finally persuaded to acquiesce in the resetting of the monument at the location from whence it had been moved. This point was fairly definitely settled by surface conditions.

Another interesting illustration of this pitfall into which a capable technician may fall is illustrated in the resurvey of the circular boundary separating the States of Delaware and Pennsylvania. This is the only mathematical curve forming a boundary that has been brought to the writer's attention. In 1849, nearly 90 years after the northeast corner of Maryland was originally monumented by Mason and Dixon, and 148 years after the original circular boundary between Pennsylvania and Delaware had been run by Tailer and Pierson, the three Commonwealths interested, by proper legislation, set up a commission to re-run and re-mark the northeastern corner of Maryland and the circular arc forming the northern boundary of Delaware.

This survey relocated the northeastern boundary of Maryland and during the excavation incident to the erection of a permanent monument, the old monument set by Mason and Dixon was found about 4 or 5 ft below the surface. The resurvey checked within about 2 in. This was well, but when the surveyor began the resurvey of the circular arc, he found some small mathematical errors in the Mason and Dixon computations and linear measurements. He surveyed the way he thought Mason and Dixon should have surveyed, following the original grants from Charles I, Charles II, and James II, and the subsequent agreements down to and including that of Frederick, the sixth (and last) Baron of Baltimore, and the heirs of William Penn. He must have done a tremendous amount of research work prior to going into the field because his report is found to be correct in every detail, and his geodetic work carefully and accurately performed. He overlooked the fact, however, that he was engaged to make a resurvey and not to establish a boundary. The result was that he took from Delaware a small triangular shaped parcel of territory, several miles long and less than 2 miles wide, and gave it to Pennsylvania. The result was that Pennsylvania, standing to benefit by the increase of territory and population, promptly approved the survey; the Maryland General Assembly not being interested in the dispute, and being contented with that portion of the survey which concerned that State's boundary, ratified the survey; but Delaware never ratified it. All the maps of this locality prior to about 1900 will show Pennsylvania (Chester County) holding this small wedge of territory extending down between Delaware and Maryland. Pennsylvania, however, was never able to collect the taxes nor in any way to exert jurisdiction on this triangle. For many years after dueling was made illegal by law, duelists sought this location for final settlement of personal disputes. Only within the past decade has jurisdiction over this tract of land been settled finally.

Maryland has a boundary dispute with West Virginia. The original grant of Maryland explicitly states that its western boundary shall be a meridian line running south from the Fortieth Parallel to the most distant (farthest from Chesapeake Bay) fountain of the Potomac River. This "most distant fountain" is known and has been known many years to be the South Branch of the Potomac River and the boundary, if properly run as originally intended, would give Maryland several counties now in West Virginia. The validity of Maryland's equity to this territory is admitted in the Constitution of Virginia adopted June 29, 1776 (Article 21), the first Constitution Virginia ever had. Due to the many Virginia patents, to the objection of many inhabitants, and somewhat to procastination, Maryland was not able to consolidate and bring this territory under its jurisdiction. In 1863, it became West Virginia. What surveyor would now make the western boundary of Maryland conform to the original intentions of the grantors and grantees?

The surveyor must bear in mind that he is not a judge. He is an expert witness to whose testimony the Court will give due weight if his work has been characterized by care and judgment. When his decisions do not conform to the law, as laid down in the decisions of higher Courts, his technical care may be of little value to his client.

C. H. EIFFERT,[98] M. AM. SOC. C. E. (by letter).—There are too many young engineers and surveyors who seem to believe that land surveying is merely a matter of turning angles and measuring distances. They are apt to place a tack in a hub with the greatest of precision but, at the same time, overlook or ignore evidences of original corners and boundary lines which, although they may not have been located accurately, are nevertheless the indicators of the true locations of the original lines. The writer would like to recommend this paper especially to these younger members of the profession.

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[98] Chf. Engr. and Gen. Mgr., The Miami Conservancy Dist., Dayton, Ohio.

It is even more difficult to convince those outside the profession that land surveying is not merely a matter of running bearings and distances given in a deed or on a plat. True, it is somewhat disconcerting, and humbling to the pride, of one who has been trained in accurate work, to find that, after measuring very precisely a recorded distance, and carefully placing the tack, it is necessary to adjust the distance 5 or 6 ft, more or less, and change the bearings similarly in order to conform to the location of a rail fence, or to that indicated by a decrepit looking bearing tree, or perhaps only the stump of such a tree. "Such is the life" of the real surveyor, however, and such difficulties should be no deterrent against "keeping the chain level, and stretched tight" and otherwise doing accurate work, because once the true line is found or re-established the accurate bearings and distances are necessary to guard against future difficulties.

It is rarely indeed that measurements made to-day with steel tape, spring balance, and correction for temperature, will agree closely with those made 50 or 75 years or more ago when land was cheap, when link chains with 600 or 800 wearing surfaces were used, and when the sight compass, not the transit, was the instrument utilized.

The writer has been told by those who claimed to be eye-witnesses that on the Western prairies surveys were made by steering with a pocket compass and measuring distances by counting the revolutions of the wagon-wheel, and that stones were simply dumped out of the wagon at the section and quarter corners. Nevertheless, right or wrong, these stones became the corners, and after the surveys had been accepted by the Government it became the legal duty of the surveyors retracing the lines to use these stones or the lines of occupation which had been etstablished in accordance with them. On such original surveys carelessly and inaccurately made, the township or other plats filed in the Government Land Office will show the miles to be 80.00 chains in length and the half-miles, 40.00 chains, and the bearings will be recorded within the required limits of accuracy; yet a resurvey will show them to be grossly inaccurate, and agreement with any of the recorded bearings or distances will be purely accidental.

A resurvey of such lines may be made with the greatest accuracy and exactly in accordance with the plat and yet be entirely wrong and inadequate if it ignores existing evidence showing where the corners were placed originally.

Too many times the surveyor, if hired by an interested party to relocate a disputed boundary line, assumes a hostile attitude toward the other party or parties in the dispute. It should be remembered that in most lawsuits arising from boundary disputes, unless extremely valuable property is involved, even the winner loses. If the surveyor can bring about the peaceful settlement of such a dispute he will be instrumental in saving money for both parties. In most cases this can be done if, after he has obtained all the facts in the case, he will explain them to the disputants and show them what the decision would probably be if the matter were taken into Court.

The writer knows of an instance in which a surveyor was employed to re-run a line in the northern woods. This he did, cutting a nice straight line and placing concrete monuments thereon. All unbeknown to him the true line, sufficiently marked so that it could be identified without a doubt, was peacefully slumbering in the woods off to one side. Other surveyors came in to lay out summer resorts, tied them to the newly monumented line, and had the plats recorded. Later, an adjoining timber owner wishing to have his timber estimated sent in a cruiser for that purpose. This man, familiar with the ways of the woods, located the original lines without difficulty. An expensive and extensive lawsuit resulted.

The re-establishment of lost or obliterated corners is an important matter and is not a job for the novice any more than are important matters in other professions. It requires common sense and good judgment and a knack of observing signs which are acquired only after considerable experience.

The U. S. General Land Office has issued a pamphlet entitled "Restoration of Lost or Obliterated Corners and Subdivision of Sections." This should be in the possession of every surveyor who may be called upon to do such work.

VERNE G. SANDERS,[99] ESQ. (by letter).—In the days when the white man started to settle the Colonies, land was plentiful, owners were few, and precise boundaries were of no importance. When towns and cities began to come into existence, neither surveyors nor land owners could visualize the future values of property, and the litigation to be involved in such a few short years. At that time there was an excuse for the poor deed descriptions and the insecure methods of boundary marking.

To-day, with the history of the almost unbelievable increase in real property values, one can look into the future with some comprehension. Now, there is no excuse for members of the Engineering Profession if they do not do all in their power to further a program of permanent establishment of property boundaries, be they across the arid desert or in the most congested districts of large cities.

The legal problems of boundary re-establishment that Professor Holt gives in his paper can be simplified, and may even be eliminated for posterity through the proper use of triangulation for controlling all surveys. By this method of control, permanent co-ordinates can be placed on each angle-point of all boundaries and filed with the deeds. Then, when re-establishment becomes necessary, all surveys for any one boundary will start at the same point, the location of which is controlled by the most accurate method of surveying known, namely, triangulation.

Several cities now use triangulation in many ways. Engineers in Los Angeles, Calif., have for many years felt the need of a precise method of control, but only in 1933, through the organization of a committee with a representative from each of the governmental agencies responsible for surveys and planning, have they been able to get a program worked out.

The first step in this program is a primary first-order net of triangulation connected to five points of the main triangulation of the U. S. Coast and Geodetic Survey. This net covers the southwestern part of Los Angeles County, an area of nearly 1 000 sq. miles, and is planned so that adequate main control is furnished over the entire City of Los Angeles.

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[99] Chf. Computer, Joint Committee on Survey Control, Through Triangulation, County and City of Los Angeles, Los Angeles, Calif.

The computations of all triangulation and traverse will be made on North American 1927 Datum. The plane system of co-ordinates as used by the Bureau of Water Works and Supply of Los Angeles for the last ten years has been adopted as a common system by all the organizations involved.

As the various steps of break-down and traverse proceed to completion, there will be one or more first-order points available for every property or street survey to be made in the County or City of Los Angeles. It is confidently expected that the legislative bodies of the City and County will pass the necessary ordinances to have all subsequent sub-divisions resurveyed, and adjudicated boundaries properly tied in and co-ordinated.

In the future this will save untold sums to property owners through the elimination of litigation, and untold sums to the taxpayer by permanently fixing the location of street and highway center lines.

A. H. HOLT,[100] M. AM. SOC. C. E. (by letter).—The considerable extent of this discussion is taken as a welcome indication of a revival, or at least of the existence, of interest on the part of engineers in the proper establishment of land boundaries. Few of the problems that come to the attention of the civil engineer in general practice possess greater potentialities for eventual importance or far-reaching effect, or call for the use of more mature and experienced judgment, than do those involving the establishment or the recovery of boundaries of land. It is time (as suggested by Mr. Rowe) that the high standards of work required for satisfactory boundary surveys should be recognized, and that work which measures up to those standards be insisted upon.

It is hoped that more States will follow the lead of Massachusetts, and a very few other pioneers, as discussed by Mr. Mueller, to a more scientific method of treating boundary problems. The establishment of a Land Court (placing this class of important, technical problems in hands especially qualified by training—legal and engineering—to rule on such cases), is a long step toward their scientific solution. The registration of title that naturally follows such assurance of competent boundary determination is a matter of great convenience in the purchase and sale of lands.

As suggested by Professor Perry, the local flavor of the paper is obvious, of course. There are two reasons for this: (1) The paper was originally presented before the Iowa Section of the Society; and (2) the apparent necessity of using the law of one jurisdiction as an example, instead of trying to treat the subject more generally, caused the writer to turn naturally to that with which he was most nearly familiar.

Quite properly, Professor Johnston emphasizes the need of setting distinctive and enduring monuments. Possibly no other one precaution will tend more directly, or more certainly, to prevent the re-opening of a boundary question than thus to express and mark its solution.

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[100] Associate Prof., Civ. Eng., State Univ. of Iowa, Iowa City, Iowa.

Second only to the importance of suitable monuments is the need of a comprehensive, reliable system of stating their positions, as a means of making possible more definite metes and bounds descriptions, of locating the monuments if they are in place, or of replacing them accurately if they have been destroyed. The obvious answer is the use of such systems of rectangular co-ordinates as are discussed by Major Bowie. In connection with the control surveys recently made in the several States under the auspices of the United States Coast and Geodetic Survey as projects of the Civil Works Administration, plane co-ordinate systems have been established for several of the States. By means of these surveys hundreds of monuments have been set and connected by traverse of second-order precision to the triangulation net. This work has served to make this means of control much more generally available. When the co-ordinates of these monuments have been determined they should, by some means, be made available for general use, and it is hoped that engineers will use them at every opportunity. The advantages of so doing are too well known to require further discussion here. That their use will meet with some opposition when that use involves others than engineers goes without saying. This should not prevent engineers from retaining the courage of their convictions and explaining and promoting their purpose. To attempt to substitute their use abruptly for time-tried methods of stating the positions of property corners would very likely result in encountering such resistance as would defeat the purpose. A metes and bounds description of land can easily be so written that all the advantages of the use of the co-ordinates will be secured, while, at the same time, it would be possible to delete all reference to co-ordinates or to co-ordinate systems and still leave a description that would be entirely acceptable under present standards. This latter feature might make such a description acceptable to abstractors, lawyers, and Courts, in the absence of a formal legalizing act. Such an act might meet with defeat if proposed, thus distinctly hindering the introduction of the use of this control.

The following is suggested as a form of metes and bounds description of land, using co-ordinates, referred to the U. S. Coast and Geodetic Survey triangulation, to define the positions of corners:

"A parcel of land situated in Blank County, Iowa, and described as follows. The co-ordinates used to define the positions of corners are referred to the system of co-ordinates established by the United States Coast and Geodetic Survey for use in the southern counties of Iowa. Bearings used are referred to the meridian of that system of co-ordinates:

"'Beginning at a point marked by an iron pin set in concrete, whose co-ordinates are: latitude 571 001.4 feet, longitude 2 461 271.3 feet, and which is on the north line of Section 00, Township 00 North, Range 0 West of the Fifth Principal Meridian, three hundred one and six tenths (301.6) feet east of the northwest corner of said section; thence south seventy-two degrees and forty-seven minutes east (S 72°–47′ E) one thousand eighty-three and four tenths (1 083.4) feet along a fence line to an iron pin set in concrete, whose co-ordinates are: latitude 570 680.7 feet, longitude, 2 462 306.2 feet; thence south sixteen degrees and twelve minutes west (S 16°–12′ W) fifteen hundred eighty-nine and three tenths (1 589.3) feet, along and in prolongation of a fence line, to an iron pin, set on the center line of Blank road, whose co-ordinates are: latitude, 569 154.5 feet, longitude 2 461 862.8 feet; thence south sixty-six degrees and five minutes west (S 66°–05′ W) twelve hundred ninety-two and one tenth (1 292.1) feet along the center line of said Blank road to an iron pin whose co-ordinates are: latitude 568 630.7 feet, longitude 2 460 681.6 feet; thence north thirteen degrees and fifty-eight minutes east (N 13°– 58′ E), to and along a fence line, twenty-four hundred forty-three and one tenth (2 443.1) feet to the point of beginning; containing forty-eight and thirty-six hundredths (48.36) acres, more or less'."

It is hoped that if attention is drawn to the characteristics of such a description noted herein, it will be possible to show to a client or to a Court the advantages of the use of the co-ordinates, and to indicate the absence of the possible disadvantage of the substitution of something that may seem new for a method that is old and time-tried, although unreliable from the standpoint of the engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1884

PHOTO-ELASTIC ANALYSIS OF STRESSES IN COMPOSITE MATERIALS

BY A. H. BEYER,[1] M. AM. SOC. C. E., AND A. G. SOLAKIAN,[2]
ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. H. J. GILKEY, ROBERT V. BAUD, AND A. H. BEYER AND A. G. SOLAKIAN.

SYNOPSIS

An analysis on models, by photo-elastic methods, of the stresses resulting from shrinkage and external loading in reinforced concrete structures, is described in this paper. The models used consist of bakelite reinforced with aluminum rods. The results that have been obtained by photo-elasticity check quite well with the accepted theory of stress analysis developed for reinforced concrete beams.

INTRODUCTION

Glass and celluloid have been used extensively for the purpose of determining the stresses in models by photo-elastic analysis. Because of its higher optical sensitivity bakelite has recently come into use. Physical tests show that this material, reinforced with aluminum, approximates the behavior of concrete, reinforced with steel.[3] At room temperatures, fortunately, the coefficient of expansion of bakelite is nearly the same as that of aluminum. The stresses in a specimen reinforced with aluminum, resulting from the application of a system of external loads, can be readily evaluated by subtracting the initial or internal stresses resulting from shrinkage and cooling, after casting, from the gross stresses developed under load. In such a speci-

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[2] Lecturer, Mech. Eng., and Research Assoc. in Civ. Eng., Columbia Univ., New York, N. Y. Mr. Solakian resigned his membership on July 9, 1934.

[3] *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 1064.

men it is also possible to make a qualitative study of the shrinkage stresses, which approximate those developed in the reinforced concrete prototype.

A number of papers were published[4] in 1932 and 1933 containing detailed descriptions of the polariscope used, the theory and application of photo-elasticity to stress analysis, the properties of the test materials, etc. In addition, Coker and Filon[5] are authorities on this subject.

FRINGE-STRESS EQUIVALENTS

In order to evaluate the stress equivalents of the isochromatic fringes—that is, the boundary lines between the color bands which are produced when polarized white light is passed through an isotropic transparent material under stress—for the particular bakelite used in this investigation, a 0.267-in. by 1.15-in. calibrating beam of the same material, cast and cured under identical conditions as the models, was analyzed in the polariscope. This beam was subjected to pure bending moments of known intensities (38.7 in.-lb, and multiples thereof). Fig. 1 is a black and white photograph of the stress fringes taken with circularly polarized white light, the green bands of the image corresponding to the black fringes in the photograph are zones of equal principal shear intensities. The average stress equivalents of the center lines of the 1st, 2d, ..., nth black fringes for the given thickness of the calibration beam, were found to be,

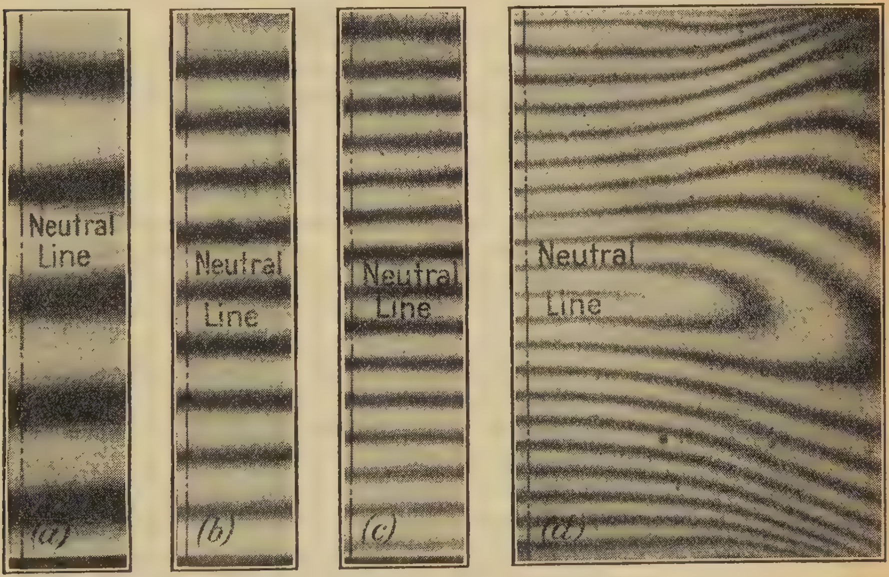

FIG. 1.—STRESS FRINGES IN A CALIBRATION BEAM IN PURE BENDING: (a) FOR $M_b = 38.7$ INCH-POUNDS; (b) FOR $2M_b$; (c) FOR $3M_b$; AND (d) FOR $4M_b$.

$$258, 2 \times 258, \ldots, n \times 258 \text{ lb per sq. in.}$$

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[4] *Transactions*, A.S.M.E. (Applied Mechanics), June 15, 1932, pp. 97-123; *Journal*, Optical Soc. of America, May, 1932, pp. 293-306; and, *Product Engineering*, April, 1933.

[5] "Photo-Elasticity," by E. G. Coker and L. N. G. Filon, 1931.

as determined from the computed values of the extreme fiber stresses of the stressed beam shown in Fig. 2.

Each group of the black and white fringes corresponds to a set of yellow, red, and green fringes as projected on the screen. From lines drawn on a sheet of paper marking the boundaries between the different color fringes, the average fringe-stress equivalents of the several isochromatics given in Table 1 were determined.

TABLE 1.—ISOCHROMATIC FRINGE-STRESS EQUIVALENTS ($t = 0.267$ INCH).

Order of color	Dividing line between:	Stress, $p-q$,* in pounds per square inch	Order of color	Dividing line between:	Stress, $p-q$,* in pounds per square inch
I.......	Yellow and red.....	185	IV.......	Yellow and red.....	875
	Red and green.....	230		Red and green.....	1 005
	Green and yellow...	305		Green and yellow...	1 110
II.......	Yellow and red.....	390	V.......	Yellow and red.....	1 150
	Red and green.....	475		Red and green.....	1 255
	Green and yellow...	575		Green and yellow...	1 365
III.......	Yellow and red.....	630	VI.......	Yellow and red.....	1 450
	Red and green.....	710		Red and green.....	1 530
	Green and yellow...	845		Green and yellow...	1 690

* p is the maximum and q the minimum principal stress and $p - q$ is equal to twice the maximum shear stress; in pure bending, $q = 0$.

BEAM REINFORCED WITH ALUMINUM

In order to be able to study stresses produced in a simple reinforced concrete element by means of models, the materials used in the composite model should have elastic properties approximating those of the reinforced concrete. Concrete and bakelite are both brittle materials. Structural steel and aluminum are both ductile. The ratio of the modulus of elasticity of steel, E_s, to that of concrete in compression, E_c, commonly designated by n, is found to vary from 10 to 15 in reinforced concrete tests. From the stress-strain curves (Fig. 3(a)) of aluminum wire in tension ($\frac{1}{16}$ in. in diameter) and bakelite in tension and compression ($\frac{3}{4}$ in. in diameter) tests, the moduli of elasticity of the two materials are found to be, for aluminum, $E_a = 9\ 700\ 000$ lb per sq in., and for bakelite, $E_b = 670\ 000$ lb per sq in.; and their ratio, E_a to $E_b = n = 14.5$, is about the same as that of steel and concrete.

The use of aluminum reinforcement for the purpose of photo-elastic stress analysis would not be satisfactory if its thermal expansion and contraction for thermal conditions at room temperature were not about the same as that of bakelite. The coefficient of expansion of an aluminum rod, $\frac{3}{8}$ in. in diameter and 12 in. long, was found to vary from 0.0000375 to 0.0000355 for temperatures ranging from 20° to 100°C (see Fig. 3(b)), while that of bakelite is found to vary from 0.000027 to 0.0000625 for the same range of temperature. Above 100°C, bakelite is quite plastic.

As a result of the difference of expansion in a specimen beam cast around aluminum rods, temperature stresses of moderate intensities are produced in both materials after cooling. Additional stresses of larger magnitudes can be set up by external loading before reaching the yield point of the bake-

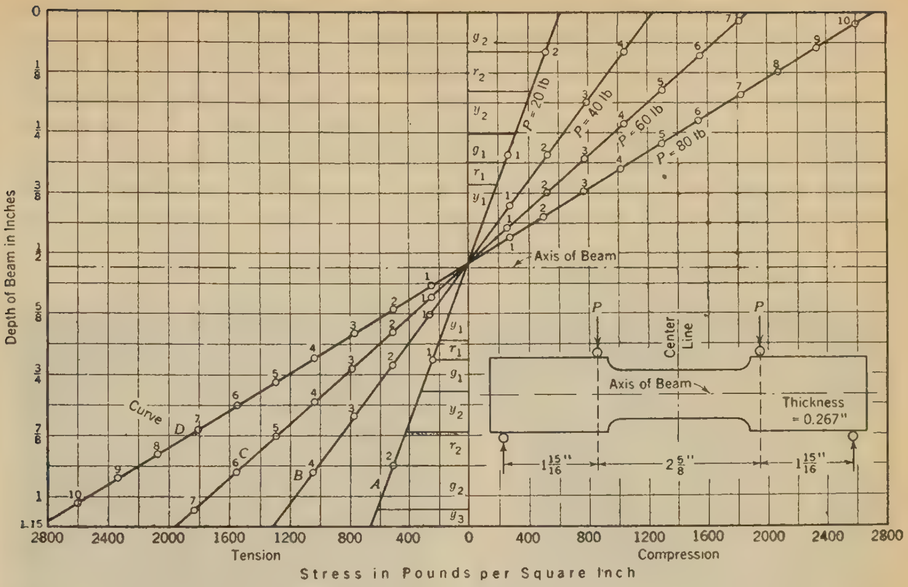

FIG. 2.—STRESS DISTRIBUTION IN CALIBRATION BEAM IN PURE BENDING.

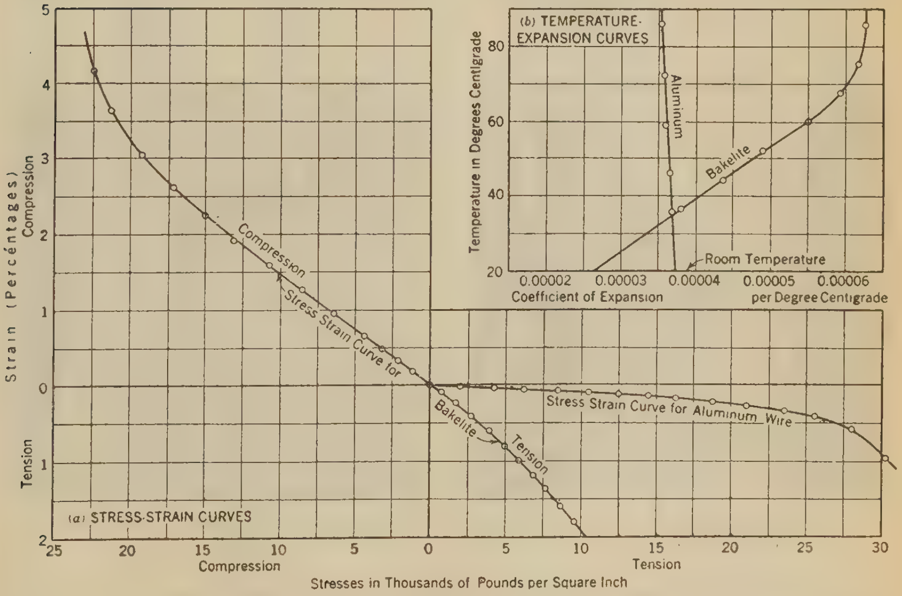

FIG. 3.—CHARACTERISTICS OF ALUMINUM AND BAKELITE: (*a*) STRESS-STRAIN CURVES; (*b*) TEMPERATURE EXPANSION CURVES.

lite. It has been found that careful annealing of specimens thus reinforced reduces the intensities of the shrinkage stresses.

Fig. 4(*a*) is a view of a fringe pattern of the initial stresses in a test beam reinforced with a piece of aluminum wire having its two ends bent at an angle. After applying a pure bending moment to the beam, the new fringe pattern, Fig. 4(*b*), appears. A complete stress analysis (with a view of the fringe pattern) of a plate reinforced with a straight bar of aluminum has been published by the writers in a discussion of another paper.[6] Fig. 13 of that discussion shows heavy stress concentrations near the ends of the rod due to shrinkage of the beam during the cooling process.

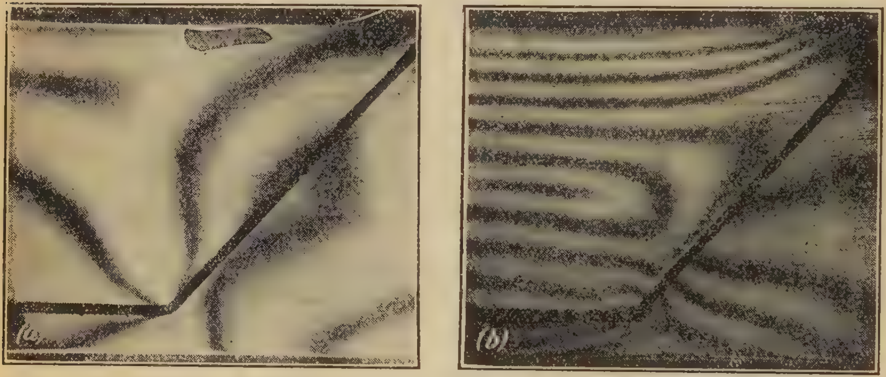

FIG. 4.—TEST BEAM REINFORCED WITH INCLINED ROD: (*a*) INITIAL STRESSES ONLY; (*b*) INITIAL STRESSES PLUS BENDING.

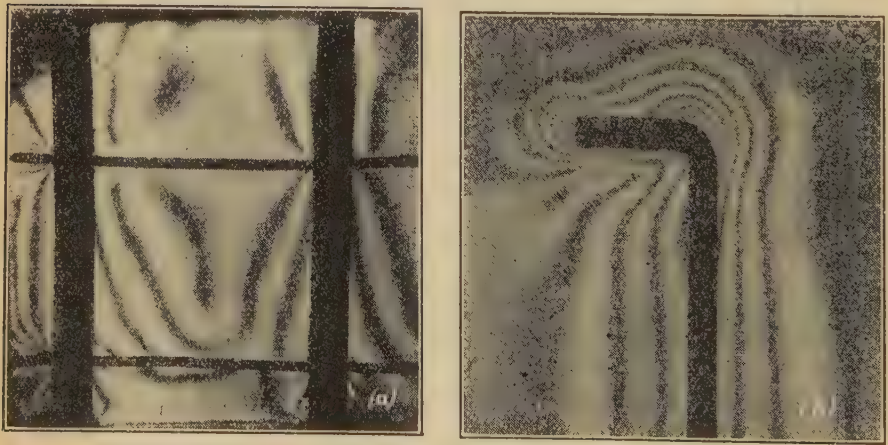

FIG. 5.—STRESS ANALYSIS OF HOOKED ROD, AND FLAT SLAB ANALYSIS: (*a*) TEST SLAB REINFORCED WITH CROSS-RODS; (*b*) ALUMINUM ROD BENT 90 DEGREES.

Fig. 5(*a*), a fringe pattern for a slab reinforced with longitudinal and transverse bars, shows the absence of material shrinkage stresses in the enclosed square, as compared to those along the edges. This is due to the beneficial effect of the transverse reinforcing bars. Fig. 5(*b*) shows the internal stress configuration adjacent to a 90° bend of a rod anchored in a plate. Note the high shear stress concentrations at the convex side of the sharp radius bend.

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[6] *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 1067.

Fig. 6(*a*) is a fringe pattern of the internal shear stresses developed in a plate similar to Fig. 5(*b*), reinforced with a rod having a large-radius bend or hook at the anchored end. On the outside of the bend the stress fringes are almost parallel to the curvature of the rod, indicating a uniform distribution of the bond stresses along that section of the rod, as compared to high stress concentration at the corresponding region of the right-angled sharp bend. After applying a pull of 100 lb to the lower end of the rod in Fig. 6(*a*), the fringe pattern obtained is as shown in Fig. 6(*b*). The variation of the maximum shear stresses along the inner and outer boundaries of

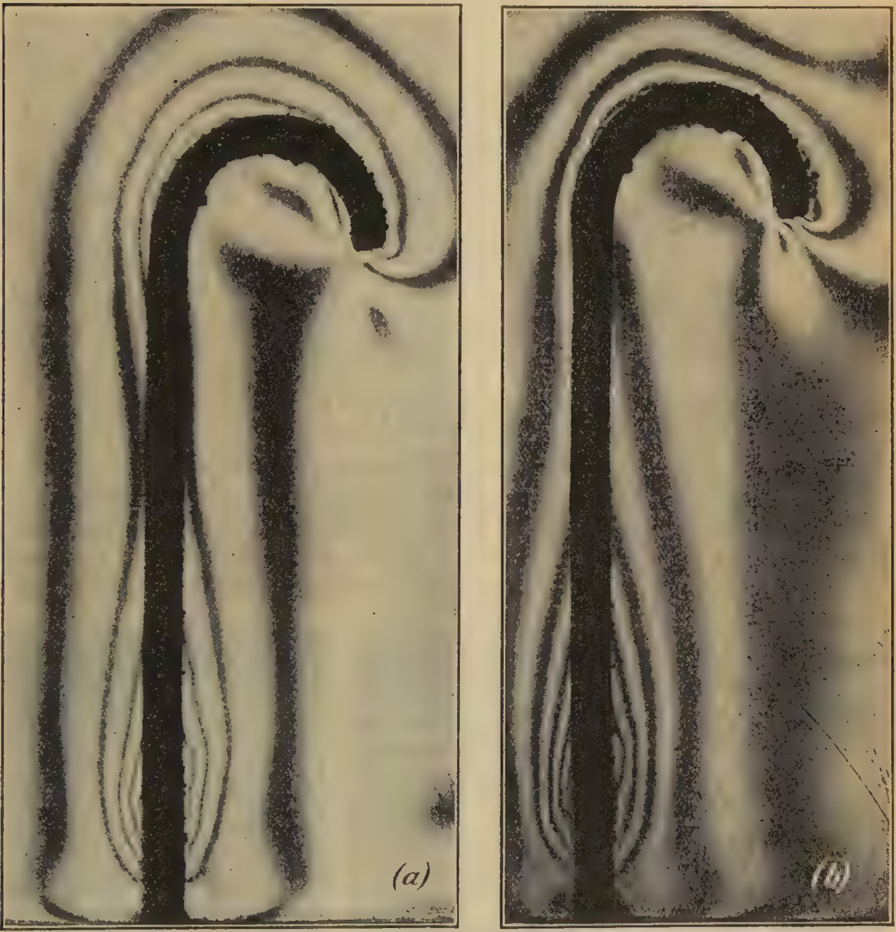

FIG. 6.—STRESS FRINGES AROUND SEMI-CIRCULAR HEAD OF ROD EMBEDDED IN PLATE; (*a*) INITIAL STRESSES; (*b*) INITIAL STRESSES PLUS PULL OF 100 POUNDS ON ROD.

the rod are given in Fig. 7, which shows graphically the initial stresses, the combined stresses resulting from the 100-lb tensile load and the initial stresses, and, finally, the net stresses due to the load alone. In all cases the highest shear stresses are found a short distance in from where the rod leaves the plate. This phenomenon may account for the cracks and crumbling off of concrete at the point where a reinforcing bar projects from a concrete mass.

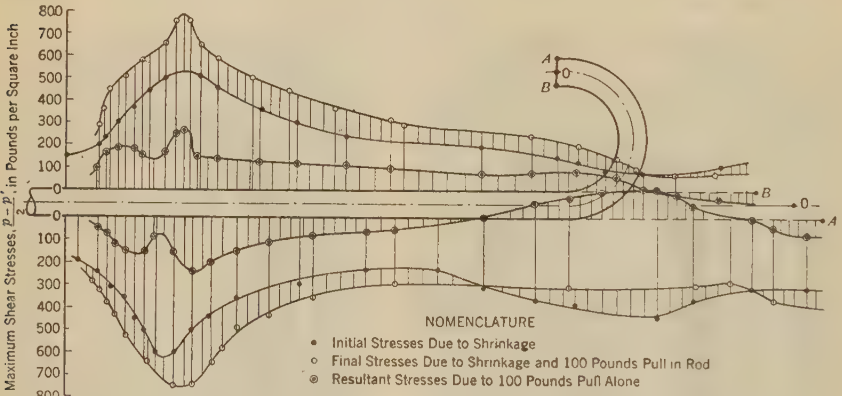

FIG. 7.—DISTRIBUTION OF MAXIMUM SHEAR ALONG THE BOUNDARIES OF A REINFORCEMENT ROD.

ANALYSIS OF A BEAM REINFORCED WITH A STRAIGHT ROD

The specimen, a beam 0.266 in. wide and 1.15 in. deep, was machined out of a bakelite casting (1 in. by 2.5 in. in cross-section). The beam had been reinforced with a rod, 0.05 in. in diameter, placed at a distance of 1 in. from the upper extreme fiber of the beam. After being polished and subsequently

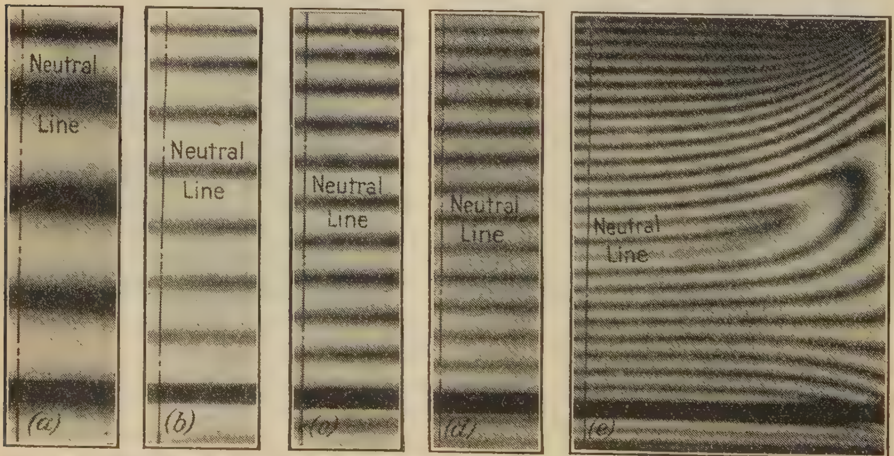

FIG. 8.—REINFORCED BEAM UNDER PURE BENDING: (a) INITIAL STRESSES; (b) INITIAL STRESSES AND M_b = 35 INCH-POUNDS; (c) INITIAL STRESSES AND $2M_b$; (d) INITIAL STRESSES AND $3M_b$; AND (e) INITIAL STRESSES AND $5M_b$.

annealed, the specimen was analyzed for shrinkage stresses and also under four different intensities of pure bending. Fig. 8 shows, in black and white, the stress fringes for the unloaded and loaded conditions. The curves in Fig. 9 show graphically the variation of these stresses over the transverse mid-section. On account of the small number of black fringes present for test conditions, $P = 0$ and $P = 20$ lb, the isochromatic fringe lines (stress equivalents from Table 1) were used to determine Curves *A* and *B*, of Fig. 9.

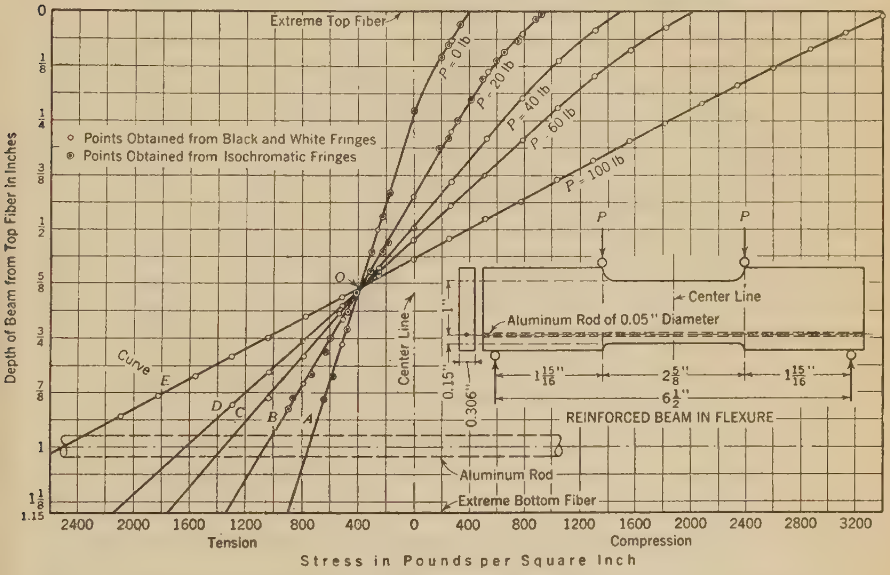

FIG. 9.—STRESS DISTRIBUTION IN A REINFORCED BEAM DUE TO INITIAL STRESS AND PURE BENDING.

For all other test conditions the black fringes (stress equivalent = 258 lb per sq in. and multiples thereof) were sufficiently great in number to locate Curves *C*, *D*, and *F*.

This analysis of the reinforced beam with initial stresses brings out the following facts (see Fig. 9):

(*a*) All the stress curves intersect at a common point, *O*, which is very close to the theoretical neutral axis. This point is 0.63 in. from the compression surface of the beam, or about 55% of the total depth. Furthermore, at this point, *O*, there exists an initial tensile stress of about 400 lb per sq in.

(*b*) In the plain beam (Figs. 1 and 2) the position of the neutral axis as determined photo-elastically coincided, for all loads, with the geometric axis of the beam. This is not the case for the reinforced beam possessing a certain amount of initial stress due to shrinkage. The apparent neutral axis for Condition *A*, Fig. 9 (no load; initial stress only), is at 0.195 *h*, in which, *h* is the total depth of the beam. As the load increases, this axis is lowered and tends to approach the true neutral axis as determined by theory. This is shown clearly in Figs. 8 and 10.

(*c*) When the internal shrinkage stresses are included the compressive stresses are not directly proportional to their distances from the neutral axis of the beam.

This phenomenon of a shifting neutral axis and a non-linear stress variation, as outlined under Facts (*b*) and (*c*), does not affect the validity of the assumptions made in the common theory of flexure of non-homogeneous materials under bending stresses. This fact is illustrated by Fig. 11 which gives the net stresses resulting from external loading alone. These stresses were obtained from the curves in Fig. 9 by subtracting the shrinkage stresses from the combined shrinkage and bending stresses. The position of the true neutral axis for the net stresses resulting only from the applied loading, as determined photo-elastically, is independent of the intensity of the loading. For the test specimen used, this axis is about 0.630 in. from the upper compressive face of the beam, 0.55 h, approximately at the common point of intersection of the curves of gross stresses (Fig. 9). The computed location of the neutral axis of this beam is 0.601 in. from the top surface, which checks closely with that determined photo-elastically. (Contrary to the common practice in the design of reinforced concrete structures, of neglecting the tensile strength of concrete in computations, the bakelite is assumed to take its full share in tension in this analysis.)

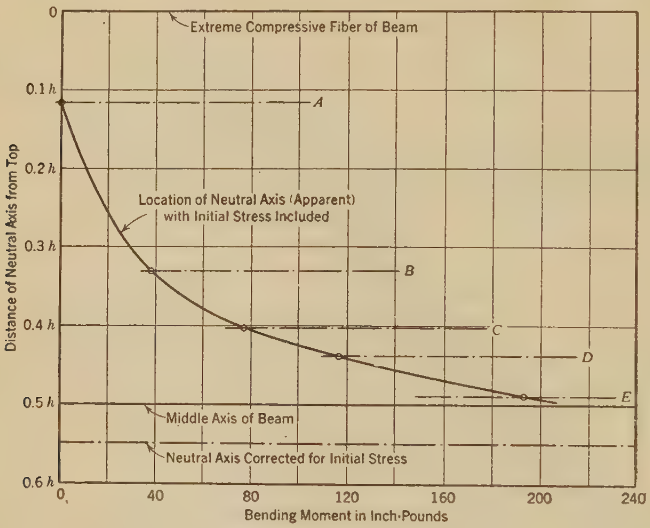

Fig. 10.—Location of Neutral Axis in a Reinforced Beam, with Initial Stresses.

It is interesting to observe that in pure bending the stresses due to loading alone are, in reinforced bakelite, directly proportional to the intensity of the bending moments. The common theory of flexure developed for non-homogeneous beams appears to apply. Using this theory and the group of curves designated by B', C', D', and E' (Fig. 11) for net stresses, the values of

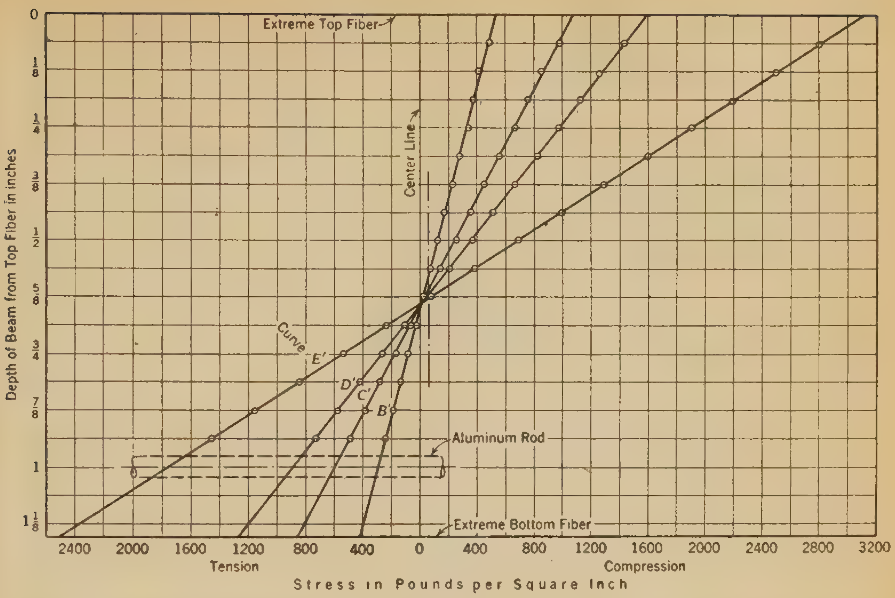

FIG. 11.—STRESS DISTRIBUTION IN A REINFORCED BEAM, DUE TO PURE BENDING ONLY.

$n = \dfrac{E_a}{E_b}$, and the intensity of stress in the aluminum rod, f_a, can be determined as follows:

In Fig. 12, let $t = 0.266$ in.; $h = 1.150$ in.; $h_1 = 0.767$ in.; $h_2 = 0.800$ in.; $x = 0.601$ in.; and, $n = \dfrac{E_a}{E_b} = 14.5$. Consider a part of the reinforced beam under a uniform external bending moment, M_b, to be in equilibrium by the internal resisting moment, M_r, of the stresses developed in the composite materials. For equilibrium conditions, therefore, $\Sigma H = 0$, or,

$$T_a + T_b = C \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

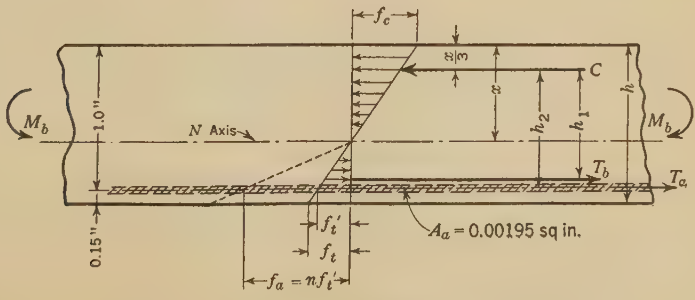
f_c
$\frac{x}{3}$
C
x
h_2
h_1
h
M_b
M_b
1.0"
N Axis
T_b
T_a
A_a = 0.00195 sq in.
0.15"
f_t'
f_t
$f_a = nf_t'$

FIG. 12.

Furthermore, $\Sigma M = 0$, or,

$$M_b = M_r = T_b\, h_1 + T_a\, h_2 \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The total tension in bakelite is,

$$T_b = \frac{f_t}{2}\,[(h - x)\,t - A_a] \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The total tension in aluminum is,

$$T_a = f_a A_a = n\, f'_t\, A_a \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The total compression in the beam is,

$$C = T_a + T_b = \frac{f_c}{2}\, x\, t \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and from Equation (4), the stress in aluminum is,

$$f_a = \frac{T_a}{A_a} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Values of f_a for different loading conditions may be computed by Equation (6) and are given in Table 2. The fiber stresses, f_t, f'_t, and f_c in Columns (3), (4), and (5), Table 2, are corrected by 45 lb per sq in. because the common point of intersection of the net-stress curves in Fig. 11 are on the compression side of the zero stress line by that amount. If values in Columns (2), (3), (5), and (9), Table 2, are plotted against values of M_b in Column (2), the relation between these factors is illustrated.

TABLE 2.—Flexure Test of a Beam Reinforced with a Straight Rod ($h_1 = 0.767$ in.; $h_2 = 0.79$ in.; $h = 1.15$ in.; $x = 0.63$ in.; $t = 0.266$ in; and $A_a = 0.00196$ sq in.)

Curve (Fig. 11) (1)	Applied bending moment, M_b, in inch-pounds (2)	Unit Stress, in Pounds per Square Inch: Maximum tensile stress, f_t, in bakelite (Fig. 11) (3)	Tensile stress in bakelite, f'_t, in the plane of the axis of the reinforcing rod (Fig. 11) (4)	Maximum compressive stress, (Fig. 11) (5)	Total Stress, in Pounds: Tension in bakelite (Equation (3)) (6)	Compression in bakelite, C (Equation (5)) (7)	Tension in aluminum, T_a (Equation (4)) (8)	Unit stress, f_a, in reinforcing bar (Equation (6)) (9)	Resisting moment, M_r, in inch-pounds (Equation (2)) (10)	Percentage error, $\frac{M_r - M_b}{M_r} \times 100$ (11)
B'......	35	455	335	500	31.0	41.9	10.9	5 590	32.4	— 8.0
C'......	70	880	635	1 030	60.0	186.3	26.3	13 480	66.8	— 4.6
D'......	105	1 290	925	1 540	88.0	129.1	41.1	21 070	100.0	— 5.0
E'......	175	2 520	1 810	3 060	171.8	256.4	84.6	43 380	198.6	+11.8

Conclusions

The photo-elastic analysis made upon the test beam described in this paper, reinforced in tension with a straight rod of aluminum and subject to pure bending, gave the following results:

(1) The stress analysis made by the photo-elastic method substantially satisfies the static equilibrium condition (see Columns (2) and (11), Table 2).

(2) The stresses in both aluminum and bakelite were found to be substantially proportional to the external bending moment for all stresses less than about 20% of the ultimate tensile strength of the bakelite.

(3) The theoretical location of the neutral axis checks closely with that determined by photo-elastic methods, but only after deducting the initial, or so-called internal, stresses.

(4) By means of photo-elastic methods the initial stresses in composite materials, and the effect of these initial stresses when superimposed upon the loading stresses, can be determined qualitatively, all of which results in a pronounced shifting of the neutral axis.

ACKNOWLEDGMENT

The writers are greatly indebted to Messrs. T. S. Taylor and J. H. Schmidt, of the Bloomfield (N.J.) Research Laboratories of the Bakelite Corporation, for their kind co-operation in preparing the reinforced specimens analyzed in this paper.

DISCUSSION

H. J. GILKEY,[7] M. AM. SOC. C. E. (by letter).—In extending the application of stress analysis by polarized light to composite materials new techniques have been developed and a new range of possibilities has been tapped. For these reasons, if for no others, the authors are deserving of commendation. A reconnaissance is usually suggestive and among the interesting further extensions that appear possible are:

1.—Studies in the transfer of stress from one bar to another as in the vicinity of a column splice. Such studies could include different types of splices and should certainly include, as a variable, the distance between the bars and the relative volume of the matrix through which the transfer of load must be accomplished.

2.—More complete experimentation with the hooked bar with reference to such factors as the radius of the hook and its relationship to the sequence of failure; that is, whether the matrix (celluloid or concrete) crushes under the bend; whether the hook "snakes" out of its curved hole; or whether it splits the block in which it is embedded.

3.—Comparison of a few large bars with many small ones to transmit a given total stress.

4.—Studies of type shapes of deformed bars, that is, those with wedge-like lugs as against those with square shouldered lugs. Such studies could also include studies of polished, etched, coated, tapered, and anchored bars.

5.—Closely allied to Item 4 would be studies of bond stresses in the vicinity of cracks and the extent to which the welded cross-wires of a mesh function as lugs.

6.—It might even be possible to study the action of hoops or spirals in resisting bursting tendencies.

7.—Perhaps future studies will develop other optical materials of properties such that the nature of the stress response can be made visible in each of the materials.

8.—Perhaps beams can be weakened on the tensile side by means of saw-cuts, or otherwise, to simulate more perfectly the tensile part of reinforced concrete members.

Whether polarized light is the logical medium to use in a search for further truth on any such problems is quite another question and should be considered carefully before an elaborate program is launched in a particular case. It must be kept in mind that:

(*a*) At best, optical methods are essentially qualitative.

(*b*) The relationship between the two materials studied differs in a number of important respects from those for the types of materials about which information is desired.

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[7] Prof. and Head, Dept. of Theoretical and Applied Mechanics, Iowa State Coll., Ames, Iowa.

(*c*) Thus far at least the evaluation of parts of the test results is by elimination, and is necessarily indirect.

(*d*) In many cases the qualitative nature of the action is already sufficiently well known, and the important fields to be covered are to obtain numerical evaluations by experimentation with members, or models of them, made up of actual materials or of materials that are similar to the actual materials.

(*e*) In other cases more thorough-going mathematical analyses will promise a more positive result and will net a greater return for the effort involved.

In spite of the questions which have been raised, the structural analyst will do well to keep these new possibilities in his consciousness, in order that he may have one more tool available if a specific need arises.

The clarity of the photographs seems to minimize one difficulty which some have recognized in considering the possible usefulness of polarized light for such studies as these. It is obvious that a stressed bar anchored by bond to a matrix, which completely surrounds it, is essentially a three-dimensional problem and, as such, would not be amenable to optical study. In other words, the bar is surrounded with successive layers or cylinders of stress. However, so much more of the cylinder walls is encountered as the edges are approached, that the direct penetration that occurs in the central part of the field offers a relatively slight obstruction to the light rays and does not appear to muddle the view as badly as it might reasonably be expected to do. Undoubtedly, however, there must be some drawing in or narrowing of the zone because of this three-dimensional aspect.

Robert V. Baud,[8] Esq. (by letter.)—In the first part of this paper the initial stresses in reinforced beams and plates are discussed. Unfortunately, the authors have not provided the bands shown in the illustrations of this part with fringe numbers, so that the views are of no practical value to the reader, because successive bands do not necessarily denote a corresponding increase in stress, such as Curve *A*, in Fig. 13, but may mean any kind of

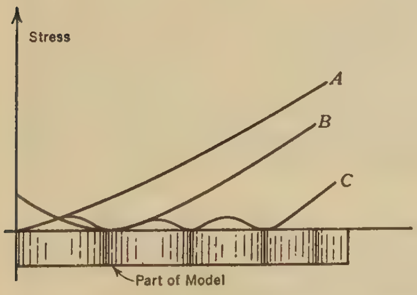

Fig. 13.

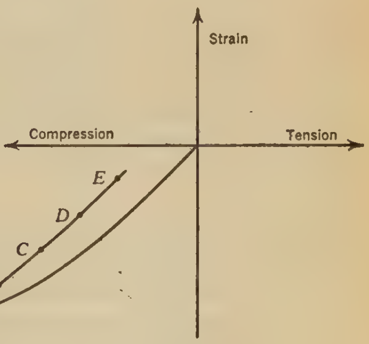

Fig. 14.

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[8] Beaver Falls, Pa.

stress distribution, such as Curves B, C, etc. The stresses in Fig. 7 are doubtful if Poisson's ratio differs for the two materials. In such a case a three-dimensional stress distribution is produced around the reinforcement rod and the stress readings are certain to be inaccurate.

It is desirable that the authors explain the percentage error in the last column of Table 2. One possible explanation is the probability that the principal stress directions for the initial condition are not all exactly parallel to the axes of the beam and that, therefore, they partly differ from the stress direction lines due to loading. If such is the case the initial field cannot be deducted algebraically, but must be subtracted according to the laws of tensor-fields. The most probable cause for the discrepancy in actual and computed moments, however, is the fact that the aluminum rod was considerably overstressed initially. The requirement that $\int_F f\,df = 0$, yields an initial compressive force in the rod of $A = 100$ lb, and the requirement that $\int_F f\,y\,df = 0$, yields a force of $A = 92.8$, or an average of about 96 lb, corresponding to an initial compressive stress of about 50 000 lb per sq in. Due to loading the beam this large initial compressive stress was reduced, but for no bending moment used in the experiment was there actually a tensile force in the rod, as is schematically shown in Fig. 14.

More important and more disturbing than the lack of agreement in the actual and computed moments (at least, for the practical man) is the fact that large initial tensile stresses are set up. Particularly for small loads the gain of shifting the neutral axis downward is more than offset by the presence of these large initial tensile stresses, and the question arises as to what this condition will be for actual materials. The authors believe that the configuration of the initial stress curve (Curve A in Fig. 9) is the same and that only the stress scale is different. If this is the case the only factor necessary to investigate further is the force, A, in the steel rod. To obtain this, one can proceed as follows: Cut a cement beam at numerous places at right angles to the rod and measure the initial and final length of the rod before and after cutting; from the difference in length the force, A, can be computed in the known manner.

As far as the optical part of the paper is concerned the writer is much pleased to note that the authors have so completely adopted, in this and other investigations, his method of simplified stress measurements.[9]

The second last sentence under "Introduction" conveys the impression that nothing of consequence has been published in this country on the subject of photo-elasticity except the papers referred to. This is misleading, because a considerable number of contributions have been made here since 1920 on the subject of the phenomena involved, as well as on the instruments used, which, as far as the measurement of the maximum shear stress is concerned, do not differ much, fundamentally, from each other.

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[9] "Further Development in Photoelasticity," *Journal*, Optical Soc. of America and Review of Scientific Instruments, Vol. 18, No. 5, May, 1929, p. 422.

From Table 1, any one not so well acquainted with the method could conclude that the colors of the orders are the same; this is not the case as a study made several years ago revealed.[10] The authors further state with regard to Fig. 1 that it "is a black and white photograph of the stress fringes taken with circularly polarized white light." This is an erroneous statement; the truth is that only one ray of the white light spectrum is circularly polarized, the others being elliptically polarized.[11] Unless the principal stress directions in model and comparison strip coincide a small error in the stress reading is introduced.

Credit is due the authors for adding a new group of problems to the already large number in which photo-elasticity proves its usefulness.

A. H. Beyer,[12] M. Am. Soc. C. E., and A. G. Solakian,[13] Assoc. M. Am. Soc. C. E. (by letter).—It was intended that this paper be considered primarily a reconnaissance in the field of photo-elasticity. Some of the possible extensions of this reconnaissance to other studies have been stated very concisely in the discussion by Professor Gilkey; but before any extensive research work can be undertaken along the lines suggested, it is essential that the investigator appreciate fully the practical applications and limitations of this new and interesting method of stress analysis as applied to engineering parts or structures.

Since 1931 a considerable part of the research work at Columbia University has been confined primarily to determining the practical applications and limitations of this method of stress analysis. The writers are convinced that photo-elastic stress analysis, when confined to uni-planar stress distributions in statistically isotropic materials, is reliable, and they have found it to check closely with those stress distributions that can be determined accurately by mathematical analysis.

The method does not lend itself to the analysis of those problems in which ultimate or partial failures occur, as outlined by Professor Gilkey. Photo-elastic methods can only be applied to those stress distributions in which the maximum stresses are well within the elastic limit of the material, because as soon as the elastic limit of the material is exceeded, the stress distribution changes materially.

As has been suggested by Professor Gilkey, the writers hope that the methods of stress analysis that have been outlined in the paper will be extended in the future to cover some of the more important problems to which attention has been called.

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[10] "The Analysis of Colors Observed in Photoelastic Experiments," by R. V. Baud and W. D. Wright, *Journal,* Optical Soc. of America, Vol. 20, No. 7, July, 1930, p. 381.

[11] "Contribution to Study of Effect of Elliptical Polarization upon Energy Transmission," by R. V. Baud, *Journal,* Optical Soc. of America, Vol. 21, No. 2, February, 1931, p. 119.

[12] Prof. of Civ. Eng., Columbia Univ., New York, N. Y.

[13] Lecturer, Mech. Eng., and Research Associate in Civ. Eng., Columbia Univ., New York, N. Y. Mr. Solakian resigned his membership on July 9, 1934.

The discussion by Mr. Baud deals largely with the technique of testing and the interpretation of the test results. He suggests that the bands shown in the illustrations should be provided with fringe numbers so that the illustrations may be of more practical value to the reader. This could have been done in the interest of clearness. It is not strictly necessary, however, because, in all important cases for the beam analyzed, the stresses increase as the distance from the neutral axis which, in all cases, has been clearly marked in the illustrations.

The data available lead to the belief that the value for Poisson's ratio is substantially the same for aluminum as for bakelite.

The comparatively large percentage of error in the last column of Table 2 is due largely to the fact that the numerical values for bending moments in the photo-elastic method, on account of the large initial shrinkage stresses present, must be obtained by elimination, an indirect method which is likely to result in small errors. Such a method is practical because the principal stresses for the two conditions are substantially parallel to each other; namely, parallel and perpendicular to the axis of the beam.

The illustrations which show the initial so-called "shrinkage stresses" in the bakelite give the stress distribution only in a qualitative way. This analysis was never intended to be a substitute for quantitative measurements such as could be readily made in a reinforced concrete structural part by obtaining the stresses in the steel from the measured strains in the reinforcing rods.

The paper was not intended to expound in detail the principles of photo-elastic technique and, for that reason, only a few references were given which dealt primarily with the application of photo-elasticity to structural engineering.

Mr. Baud takes exception to the statement in the paper that Fig. 1 is a black and white photograph of the stress fringes taken with circularly polarized white light. In this he is correct; only one ray of the white light spectrum is circularly polarized, the others being elliptically polarized. In taking the photograph, a filter was used to eliminate all except the green light, so as to produce a more nearly perfect circularly polarized light than is possible with white light, without using a filter. Furthermore, the elliptical polarization effect probably introduces no errors of sufficient magnitude to detract from its practical value.

In conclusion, the writers wish to thank the discussers for their valuable criticisms, especially those of Mr. Baud, who (as stated in his discussion) has published some very important papers dealing with both the technical and optical aspects of the use of polarized light in determining the stresses in transparent materials.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1885

DURATION CURVES

BY H. ALDEN FOSTER[1], M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. RICHARD PFAEHLER, EDWARD H. SARGENT, C. R. PETTIS, W. G. HOYT, ADOLPHO SANTOS, JR., JOHN D. WATSON, HOWARD L. COOK, C. S. JARVIS, DONALD H. MATTERN, F. KNAPP, DINO TONINI, AND H. ALDEN FOSTER.

SYNOPSIS

The method of analyzing statistical data by means of the duration curve is discussed in a general way in this paper. The relation of the duration curve to other statistical curves is described. The computation and methods of plotting the curve are outlined, and its mathematical and graphical characteristics are explained and illustrated, followed by a description of graphical methods for using it. Examples of its application to hydro-electric power problems have been included and worked out in detail. The paper concludes with a brief historical review.

INTRODUCTION

Of the various graphical methods that have been developed for the study of statistical data, those most familiar to hydraulic engineers are probably the hydrograph, the mass curve, and the duration curve. The first two have been used for many years and are fully described in many textbooks. The duration curve, however, has only been in general use since about 1915. Although it is described in a few recent texts on water-power engineering, there are many matters connected with it which have never been adequately explained in print, as far as the writer has been able to ascertain.

It is the purpose of this paper to describe the general properties of the duration curve, and to present certain applications to hydraulic and water-power problems. The writer does not claim credit for originating the methods of using the duration curve outlined herein, but has tried to give credit for particular methods wherever the source of these methods has been known.

NOTE.—Published in October, 1933, *Proceedings*.

[1] Res. Engr., Parsons, Klapp, Brinckerhoff & Douglas, North Platte, Nebr.

His aim has been to describe as briefly as possible the methods used in the office of the firm with which he is associated, with additional comments based on his own study and experience. He has found that those who have not had much experience in the use of duration curves often encounter difficulty through ignorance of the fundamental principles involved. It is hoped that the discussion will bring out additional practical uses of the duration curve.

Graphical Methods for Studying Stream Flow

The simplest method for analyzing stream-flow data is by the hydrograph, Fig. 1(*a*), which shows the monthly run-off of the Cedar River, at Cedar Rapids, Iowa. This is merely a direct graphical plotting of the statistics, the abscissas showing the successive days, and the ordinates the flows occurring on those days. The most important characteristic of the hydrograph is that it presents the statistical data in their true chronological sequence.

The mass curve was first suggested by W. Rippl, in 1882[2]. It is obtained (see Fig. 1(*b*)) by summing the flows of the successive days, weeks, or months, and, hence, gives the total volume of water that has flowed down the stream since the beginning of the record. The mass curve, therefore, is equivalent to the integral of the hydrograph, if the latter could be considered as a continuous mathematical function; and the hydrograph is the derivative of the mass curve. The mass curve represents the area under the hydrograph. The slope of the curve at any point represents the rate of flow at that time. This curve is useful in analyses of stream flow, particularly in studying storage requirements and regulated flows. Its characteristics and uses are well described in many textbooks.

Another method of analyzing the hydrograph, which has long been in use by statisticians, is by the frequency curve, which shows the number of items in the data that have any given magnitude, or the magnitude of which falls within certain limits. Fig. 1(*c*) shows the frequency curve of monthly stream flow on the Cedar River, at Cedar Rapids, for the years 1903 to 1923. Due to the limited number of items, this is plotted as a block diagram rather than as a continuous curve. It shows the number of months in the total period of record (or the frequency of occurrence of the months) in which the stream flow lies between certain limits. For example, during 20 months of the total period, the monthly stream flow was between 3 000 and 3 500 cu ft per sec.

Unless the original data are very numerous, the frequency curve is likely to be rather irregular. Moreover, it does not bear any direct mathematical relation to the hydrograph because the data are not shown in any true chronological order.

If the values shown by the frequency curve are added successively, the "cumulative frequency," or duration curve, is obtained. This curve shows the total number of items in the data which are smaller (or larger) than any given amount. Fig. 1(*d*) gives the duration curve (diagram) corresponding to the frequency diagram of Fig. 1(*c*). It shows the number of months in the period of record in which the flow was equal to or

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[2] *Minutes of Proceedings*, Inst. C. E., Vol. 71, p. 270.

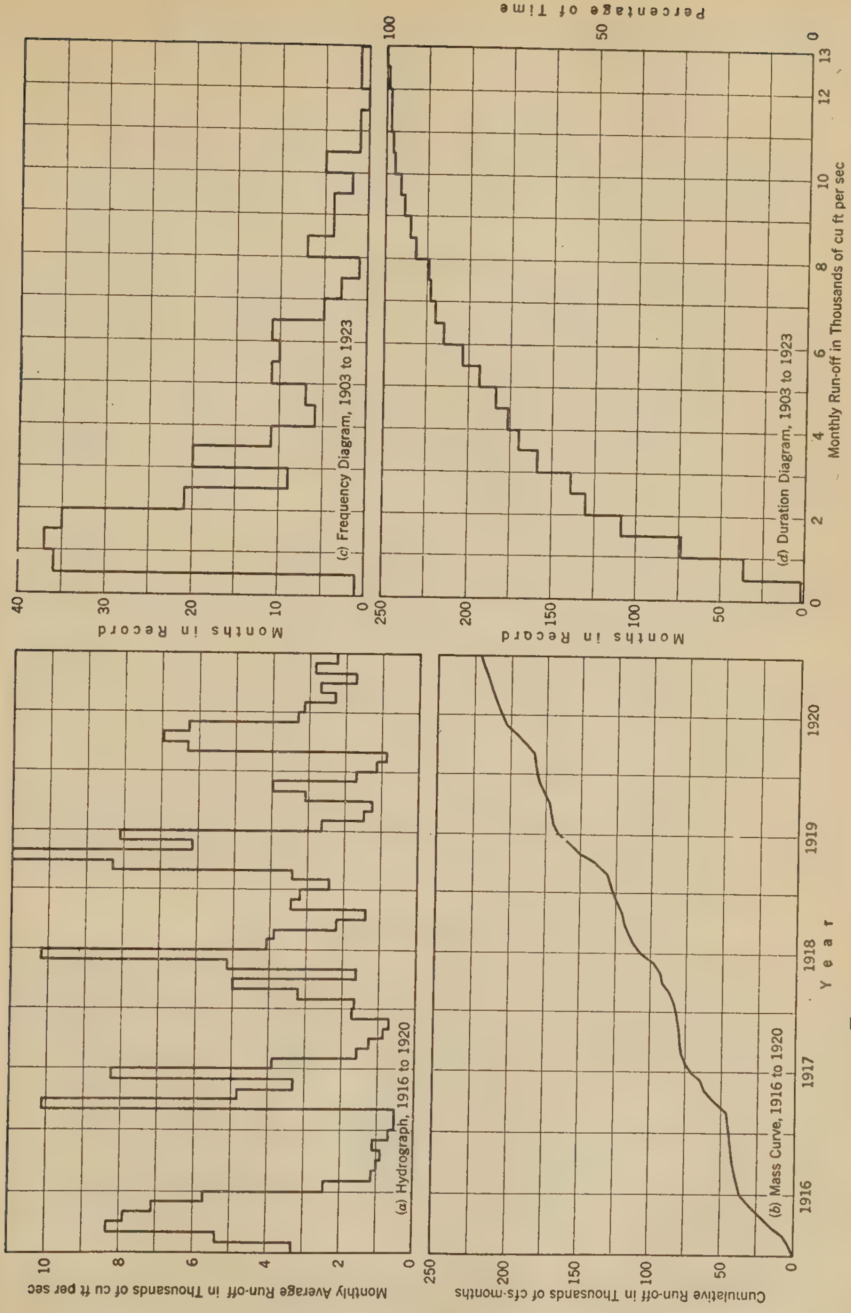

FIG. 1.—MONTHLY AVERAGE RUN-OFF, CEDAR RIVER, AT CEDAR RAPIDS, IOWA.

smaller than any given rate. If the total number of time units in the record is taken as 100%, the curve will show the "percentage-of-time" that the flow is equal to or less than any given rate. Such curves therefore, are sometimes referred to as "percentage-of-time," or "%-of-time," curves.

The duration curve is much more regular in shape than the frequency curve, even with only a relatively limited number of items in the data. Since in most engineering work the data are not very extensive, the duration curve will generally give a more convenient picture of the statistics than is obtained from the frequency curve.

If the curve is placed so that the %-of-time scale is the axis of ordinates (as in Fig. 1(*d*)), the duration curve may be considered as the integral of the frequency curve, and the latter is the derivative of the duration curve. It is generally more convenient, however, to use the %-of-time axis for abscissas and let the ordinates show the numerical values of the original data.

Frequency and duration curves may be considered as forms of probability curves, showing the probability of occurrence of items of any given magnitude in the data. Methods have been developed for representing such data by theoretical equations or curves.[3]

The duration curve, as constructed from actual records, may have two distinct uses: (1) If treated as a probability curve, it may be used to determine the probability of occurrence of future events; this use in hydraulic problems was first proposed[4] by the late Allen Hazen, M. Am. Soc. C. E.; and (2) it can also be used separately from its character as a probability curve, merely as a convenient tool for study of the data, just as the hydrograph or mass curve is treated. It is to this use of the duration curve that the present paper is limited.

In the following discussion, the use of the duration curve is illustrated by application to problems in stream flow. However, these methods are general in their application, and may be used with any statistical data that can be arranged as a frequency distribution. They should be useful, therefore, in other branches of engineering work.

CONSTRUCTING THE DURATION CURVE

The simplest form of duration curve is constructed by listing all the items in the data under consideration in their relative order of magnitude (either increasing or decreasing). The diagram on which the curve is to be plotted is then divided into vertical segments of equal width, the number of segments being equal to the total number of items in the data. Each item is laid off to the proper vertical scale in order of magnitude, in its proper segment. A horizontal line is drawn across each segment at the point thus noted, the result forming a "block diagram" or "histogram" as shown in Fig. 2, which is the duration curve of annual run-off of Cedar River, at Cedar Rapids, from 1903 to 1923.

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[3] "Theoretical Frequency Curves and Their Application to Engineering Problems," by H. Alden Foster, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142. For discussions of the use of duration curves in probability studies by other writers, see *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 191, and Vol. 91 (1927), p. 1.

[4] *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

If the center points of the tops of all blocks of such a diagram are joined by straight lines, a duration "curve" will be obtained, as shown by the broken line in Fig. 2. Each item of the duration series is then plotted at the mid-point of its respective sub-division of the diagram.

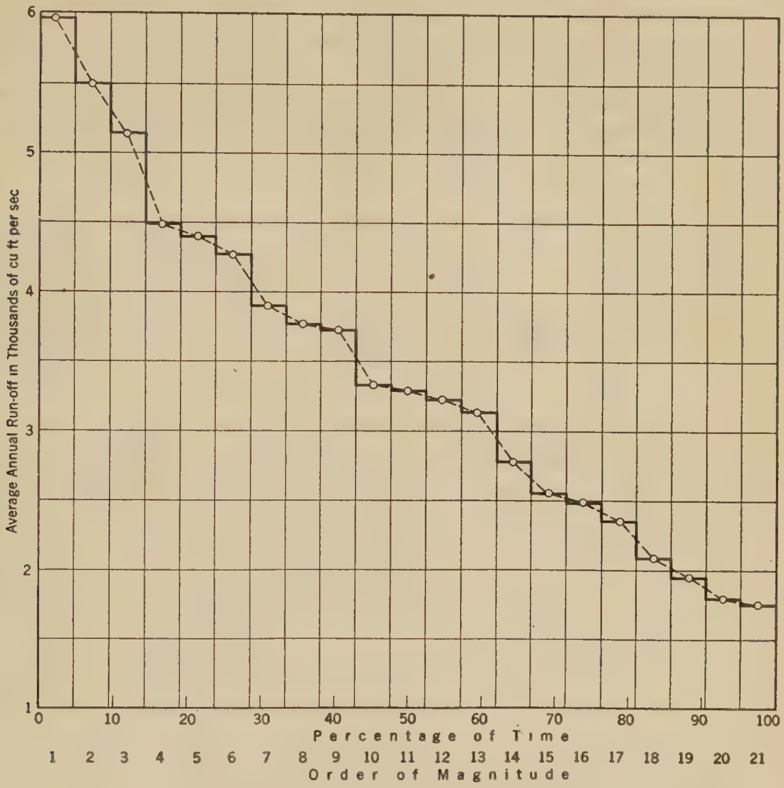

FIG. 2.—DURATION CURVE: AVERAGE ANNUAL RUN-OFF, CEDAR RIVER, AT CEDAR RAPIDS, IOWA, 1903 TO 1923.

Thus far, the "time" scale of the duration curve (axis of abscissas) has been subdivided into the same number of parts as there are separate items to be plotted. It is generally more convenient to plot these items on a percentage scale or %-of-time basis. For this purpose the total number of items to be plotted is represented as 100 per cent. If there are n items in all, each sub-division of the diagram will have a width of $\frac{100}{n}$ per cent. Then, the first item will be plotted at the center of the first segment, or at $\frac{100}{2n}$ %; the second, at $\frac{100}{2n} + \frac{100}{n}$, or at $100\left(\frac{3}{2n}\right)$, etc. If the relative position of any item is at the mth segment, its plotting position will be $\frac{100\,(2m-1)}{2n}$ on the %-of-time axis. Such a %-of-time scale is also shown on Fig. 2.

When there are many items in the original data, the labor of tabulating and plotting becomes very great; also, the space between adjacent values on the %-of-time scale becomes too small for accurate plotting. It is then more convenient to group together all items that lie between certain assumed limiting magnitudes, and plot them as one point on the duration curve. The plotting position would then be at the %-of-time corresponding to the middle item of the group, and its ordinate should equal the average magnitude of the items in the group. If the limits selected for the group are not too far apart, this average may be replaced by the average of the limiting magnitudes.

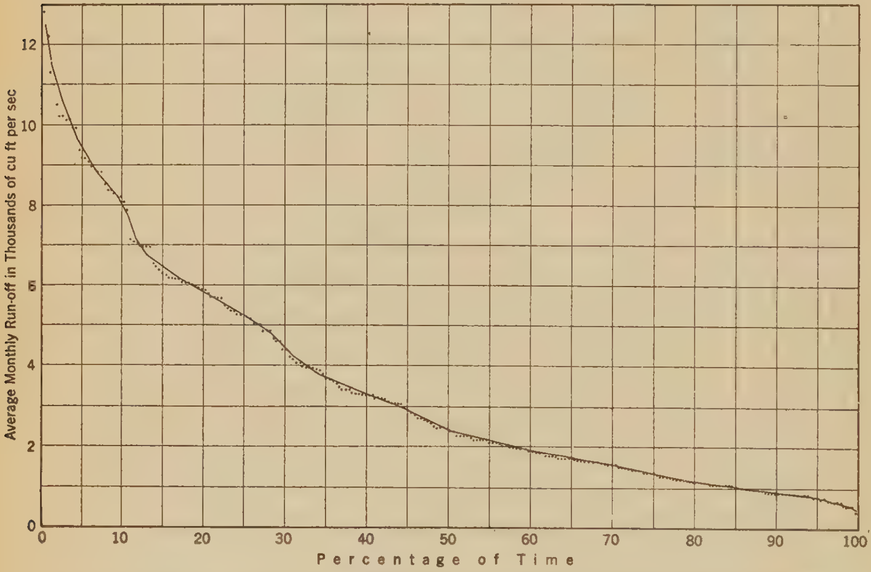

FIG. 3.—DURATION CURVE: AVERAGE MONTHLY RUN-OFF, CEDAR RIVER AT CEDAR RAPIDS, IOWA, 1903 TO 1924.

This method of plotting is illustrated by the solid line in Fig. 3, the computations for which are indicated in Table 1. It should be noted that twenty-two lines are omitted in Table 1, for brevity. The values in Column (4) are obtained by subtracting one-half the number of items in the group from the summation of the number of items and adding 0.5; that is, *m* equals the numerical order of the center of the group, plus one-half.

Fig. 3 also shows the plotting of the individual monthly values, in order to give a comparison between the two methods of plotting. The method of grouping the items involves only a very slight loss in accuracy if discretion is used in selecting the size of the groups; and the labor involved is greatly reduced.

When individual items are plotted, as shown in Fig. 2, the sum of the plotted ordinates will equal *n* times the average of all the items. From this it follows that, if the %-of-time method of plotting is used, the total area under the duration curve will equal 100 times the average ordinate, or 100 times the average of the recorded values. Similarly, the area under any part of the curve may be considered to represent the average of the recorded values included in that part.

TABLE 1.—MONTHLY AVERAGE RUN-OFF, CEDAR RIVER, AT CEDAR RAPIDS, IOWA (Record of 1903 to 1924; $n = 250$)

Run-off in order of magnitude (cubic-feet-per-second limits) (1)	Number of items (2)	Summation of number of items (3)	PLOTTING POSITION	
			Numerical $= m$ (4)	Percentage $= \frac{100\,(2m-1)}{2n}$ (5)
13 000–12 000..................	2	2	1.5	0.4
12 000–11 000..................	2	4	3.5	1.2
11 000–10 000..................	5	9	7	2.6
10 000– 9 000..................	6	15	12.5	4.8
9 000– 8 500..................	5	20	18	7.0
******************************	************	**************	**************	***************
800–700....................	5	240	238	95.0
700–600....................	5	245	243	97.0
600–500....................	4	249	247.5	98.8
500–400....................	1	250	250	99.8

The method of plotting at the center of each group involves certain principles which should be kept in mind. The only theoretically accurate way to plot a limited number of items is by a block diagram. When an attempt is made to represent the data by a continuous line, it is automatically assumed that there are an infinite number of items to be plotted; that is, from a mathematical point of view, a continuously varying function has an infinite number of possible values. If the data were increased in quantity, there would be several values to plot between each successive pair of items on the original diagram. For example, if in a 5-year record of monthly stream flow

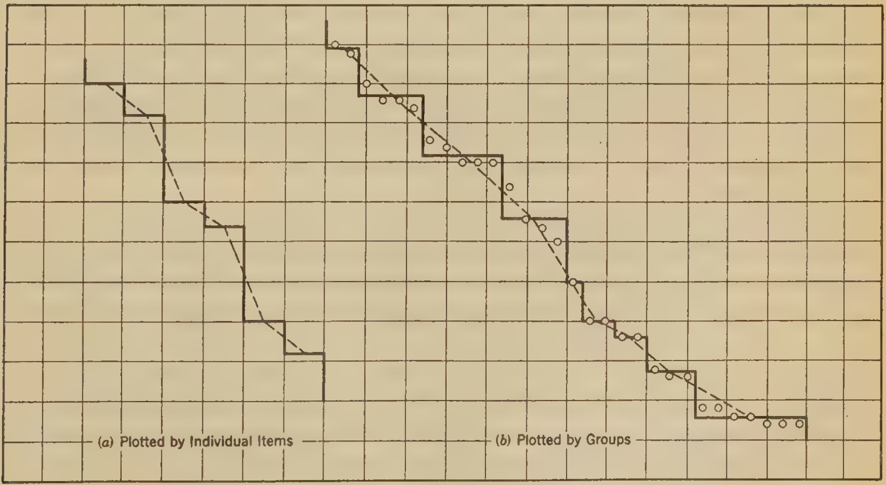

FIG. 4.—TYPICAL SECTIONS OF A DURATION CURVE.

there are no recorded values between 2 500 and 3 000 cu ft per sec, one would expect to find one or more monthly flows between these limits if the record were extended to cover 20 or 30 years. A record of infinite length, having an infinite number of items to be plotted, would result in a smooth curve instead of an irregular block diagram. Having only a limited number of items from which to plot a continuous curve, the assumption is made that this continuous curve would pass through the center of each block of the actual record.

When each item of the record is plotted (Fig. 4 (*a*)), the area under the section of the continuous curve within the limits of any part of the diagram will be equal to the corresponding area under the original block diagram. Hence, results obtained from the two diagrams will be in exact agreement. When the items are "bunched" in groups of various sizes, so that the "blocks" of the diagram have variable widths, as shown in Fig. 4(*b*), the curve plotted through the center points of the successive blocks will show a slight discrepancy in area as compared with the original block diagram at any given point; but these errors are compensating. Moreover, the continuous line obtained in this way gives a more satisfactory representation of the complete data than the block diagram obtained by the grouping method.

In general, any method of plotting is satisfactory provided the area under any section of the resulting curve is the same as that under the corresponding section of the block diagram plotted from the complete data.

Plotting in Terms of Mean Flow

When the run-off as recorded at a given point on the stream is to be compared with the record at another point on the same stream, or when the records on different streams are to be compared, it is desirable to divide the original stream-flow values by the average run-off at the point of record, this average being obtained from a record of as great a length as possible. A duration curve constructed from this reduced record will give the flow "in terms of mean run-off," and will reveal as much detail as one obtained from the original data, but in a more convenient form for comparison with other duration curves.

The run-off record at a certain point, *A*, on the stream may be used to represent conditions at some other point, *B*, on the stream where there is no available record, by plotting in terms of mean run-off. Available run-off, power, or other results obtained from this reduced record may be multiplied by the estimated average run-off (power, etc.) at *B*, in order to obtain corresponding results at Point *B*. This procedure involves the assumption that the run-off for simultaneous periods at the two points is proportional to the long-term average run-off at these points. Such an assumption is not necessarily true at all times; but it may form the only feasible basis for an estimate of stream flow at a point where no records are available. Of course, such a transfer of records from one point to another should be used with caution, especially when there is a considerable difference in drainage area or topography of water-shed at the two stations.

Fig. 5 shows the duration curves of run-off at two points on the same water-shed, the flow being expressed in terms of the average run-off in each case. This curve affords an interesting comparison between the stream flow at points of greatly different drainage area and average run-off. The Rogersville record refers to a drainage area of 3 060 sq miles at Rogersville, Tenn., on the Holston River. The average run-off in this case was 4 247 cu ft per sec. The Butler record refers to a drainage area of 427 sq miles at Butler, Tenn., on the Watauga River. The average run-off in this case was 708 cu ft per sec. The length of record is also quite different; but the resulting duration curves show a relatively close agreement as to their general shape, thus indicating that the curve for one point could be used to approximate conditions at the other point. Such a relationship would be difficult to determine from the original records, due to the great difference in the actual flows at the two points.

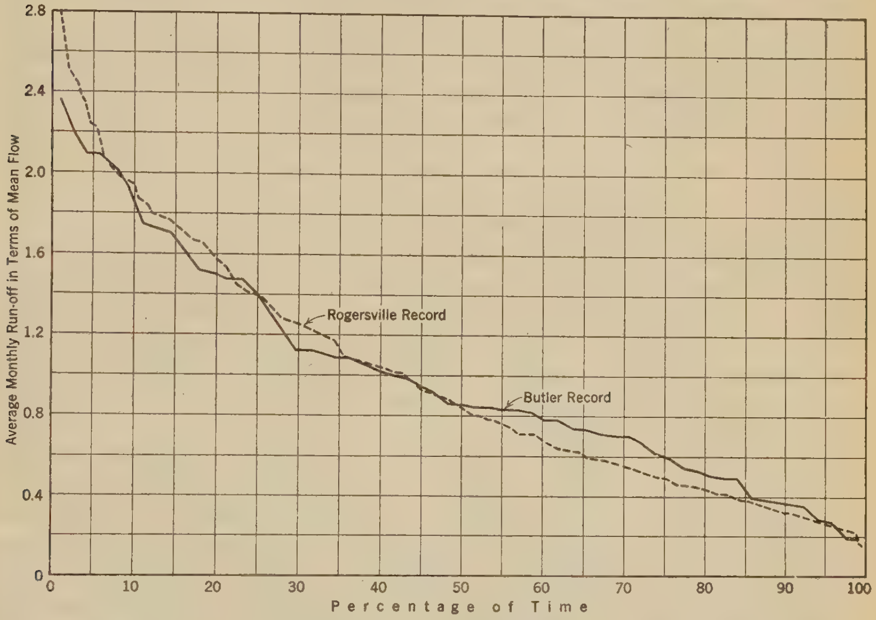

FIG. 5.—DURATION CURVES: AVERAGE MONTHLY RUN-OFF IN TERMS OF MEAN FLOW.

UNITS OF MEASUREMENT

With any duration curve of stream flow, since the ordinates represent the rate of stream flow and the abscissas show the time, the area under any part will represent the volume of flow occurring during the periods of time covered by that part; and the total area represents the total volume of run-off during the entire period of record. When the curve is plotted in terms of the mean stream flow and on the %-of-time basis, the total area under the duration curve must equal 100 times the average flow; or, since the average is taken as unity, this area will be 100.

If one linear unit on the horizontal axis represents 100 p %-of-time, and one linear unit on the vertical scale represents q units of stream flow, then one unit of area represents pq volumetric units of run-off. For example, suppose 1 in. on the time scale equals 10% ($p = 0.1$), and 1 in. on the vertical scale equals 0.4 times the average run-off; then 1 sq in. represents 0.04 times the total volume of run-off during the entire period of record. If the duration curve is assumed to represent the conditions during an average year (100%-of-time equals 1 yr), and the average stream flow is 708 cu ft per sec, then 1 sq in. equals $0.04 \times 708 = 28.32$ cfs-yr (cubic-feet-per-second-year), or a total volume of $28.32 \times 31\ 536\ 000 = 893\ 000\ 000$ cu ft. (A cubic-foot-per-second-year is the total volume produced by a stream flow of 1 cu ft per sec continuing for 1 yr; that is, 31 536 000 cu ft.)

In general, when the flows are plotted in terms of the average flow, and the average stream flow is s cu ft per sec, one unit of area will represent $p\ q\ t\ s$ cfs-yr, in which 100%-of-time is considered to equal t yr—or $31.536 \times p\ q\ t\ s$ million cu ft. In the foregoing example, if the curve represents conditions during 1 month, then 1 sq in. equals $0.1 \times 0.4 \times \frac{1}{12} \times 708 = 2.36$ cfs-yr, or $2.36 \times 31\ 536\ 000 = 74\ 400\ 000$ cu ft.

This relation of the units holds true regardless of the period of time covered by the individual items of the original data. Thus, the record may give the average stream flow of each day for one year, in cubic feet per second. If the %-of-time basis is used for the plotting, the total area under the curve will represent the total annual run-off, in cubic-feet-per-second-years; and each unit of area will represent a certain volume of run-off which will also be in cubic-feet-per-second-years.

It may be convenient in some cases to reduce the original data to some function of the stream flow before plotting the curve. Thus, if a study is being made of the power available at a given site, the stream flow (in cubic feet per second) can be reduced to kilowatts corresponding to the net head and plant efficiency available at the site. The duration curve will then show the average kilowatts available during each day, week, or month, according to the time unit used in tabulating the stream-flow data. If the %-of time basis is used for plotting, then one unit of area will equal $p\ q\ t$, in kilowatt-years, in which, one linear unit on the %-of-time axis equals $100\,p$ %, one linear unit on the vertical axis equals q kw, and 100%-of-time equals t yr. If the ordinates give the power in terms of the average kilowatts available (this average being equal to K), and one unit on the vertical axis represents q times the average kilowatts, then one unit of area equals $p\ q\ t\ K$, in kilowatt-years.

In any case, the total area under the curve, when multiplied by the numerical factor corresponding to the scale units used in plotting the curve, should equal the total "output" of the stream during the period of record, in cubic feet, kilowatt-hours, etc., depending on the units in which the data are expressed.

INFLUENCE OF TIME UNIT FOR ORIGINAL RECORDS

Stream-flow records are generally given in the form of average flow for each successive 24 hours. As has been pointed out, the use of data in this form involves an immense amount of work. It is often convenient to average these data for successive weeks, months, or years. (The United States Geological Survey records are generally given in both daily and monthly form.) Obviously, any lengthening of the time unit used for each item to be plotted in the duration curve will result in a loss of detail, since the flows will not generally be constant over the period of time for which they are averaged. It is important, therefore, to know what the relative effect will be on the accuracy of the duration curve caused by such changes in the time unit.

In Fig. 6 are shown the daily, monthly, and annual duration curves of stream flow obtained from the same record. These are typical of all duration curves plotted from the same record, but with different time units. As the time unit is increased in length, the lower end of the duration curve will rise and the upper end will be lowered on the diagram. This is to be expected, since the averaging process involved in using a longer time unit must result in eliminating many of the smallest and the largest items in the record.

The total area under each of the curves in Fig. 6 is the same, because this area represents the total volume of run-off during the entire period of record, and this volume is not affected by changing the time unit.

The relative effect of varying the time unit will not be the same with all streams. Where the flow is not subject to any sudden changes, as on the St. Lawrence River, it will be almost constant over considerable periods of time. With such a stream, the daily and weekly duration curves would be almost identical; and the monthly duration curve would not differ greatly from the daily curve. On the other hand, if the stream is "flashy," with sudden floods lasting only a few hours, or days, there will be an appreciable difference between the daily and weekly curves; and the monthly curve will involve a considerable percentage of error as compared with the daily curve.

Unfortunately, it is often necessary to use the monthly instead of the daily or weekly curve, for two reasons: (1) With some streams, only the monthly average flows are available over part or over all of the period of record; and (2) in most preliminary investigations, the time and labor required to prepare the daily or weekly curves are so great that a longer period must be used. In any case, when the monthly curve is used, its limitations should be recognized; and if the stream is at all "flashy," the results should be properly discounted when estimates of plant capacity and output are being made. These comments should apply with equal force to other methods for studying stream-flow data, such as the hydrograph or the mass curve.

The Duration Curve Represents Average Conditions

It is often convenient to consider the duration curve constructed from a given record as representing the average year of that record. Strictly speaking, there is no such thing as an "average year." There may be no particular year in the period of record during which the annual stream run-off, seasonal distribution of flow, maximum and minimum flows, etc., could all be considered as averages of the entire record. The expression is really an abbreviation. When reference is made to the kilowatt-hour output of a hydro-electric plant in an "average year," the meaning is the total number of kilowatt-hours that could be produced during the entire period of record, divided by the number of years of record. This might also be referred to as the "average annual output."

It is in representing such average results that the duration curve is often most useful. The area under any section, if multiplied by the factor, $p\ q\ t$, will give the volume of run-off represented by that part of the diagram. If the stream flow is given in cubic feet per second, and the number of years of record is t (= 100%-of-time), this volume is expressed in cubic-feet-per-second-years. If this volume is then divided by t, the result will be the average volume per annum (in cubic-feet-per-second-years) represented by that part of the diagram. In other words, the area of a given part of the diagram multiplied by $p\ q$ gives the average annual run-off represented by that part; and the total area under the duration curve represents the average annual run-off of the stream.

The duration curve constructed from a record covering t years not only shows the conditions for all those years, but also may be considered to represent the average annual conditions for the t years. On this basis, it may be stated that the duration curve does represent conditions in the "average year," as indicated by the given record. This is a useful point of view, because it enables the engineer to forget about the length of the original record, and to treat the curve as representing one complete year that is typical of the entire record. Moreover, on the assumption that average results in the future will be similar to those obtained from past records, the same duration curve can also be treated as representing the average year in the future.

In like manner, the duration curve may be considered to represent an average month. In this case, if the ordinates show stream flow, in cubic feet per second, the total area under the curve will represent the average monthly run-off (in cubic-feet-per-second-months), or the total run-off in an "average month."

In some cases, it may be desirable to construct a duration curve for a record of less than one year. Such a curve would be a true representation of conditions for that particular period, but should not be treated as representing conditions for an average year or an average month.

Regulated Flow and Storage

In many problems, it is desirable to construct a duration curve of the flow of a stream (or of some function of the flow, such as the kilowatt capacity of the power house) for conditions after the stream has been regulated by an assumed storage reservoir The original stream-flow record will furnish a duration curve of "natural flow," before regulation is begun. The same record is then revised to correspond with the assumed method of regulation, and from this is derived a new duration curve of "regulated flow," indicating conditions as they would have existed during the actual period of record if the assumed regulation program had been in effect.

The relation between the duration curves for natural and regulated flow is shown in Fig. 7. The method of regulation illustrated in this diagram is to

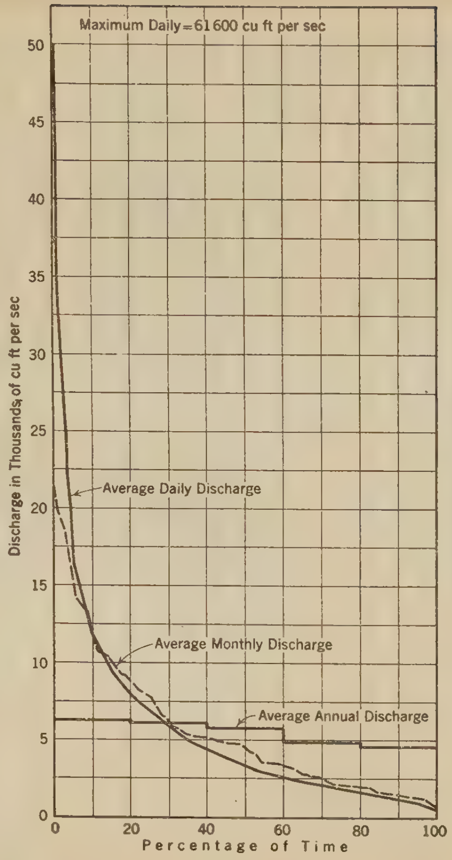

FIG. 6.—DURATION CURVES OF AVERAGE DAILY, MONTHLY, AND ANNUAL DISCHARGE, DELAWARE RIVER, PORT JERVIS, N. Y., OCTOBER 1, 1916, TO SEPTEMBER 30, 1921.

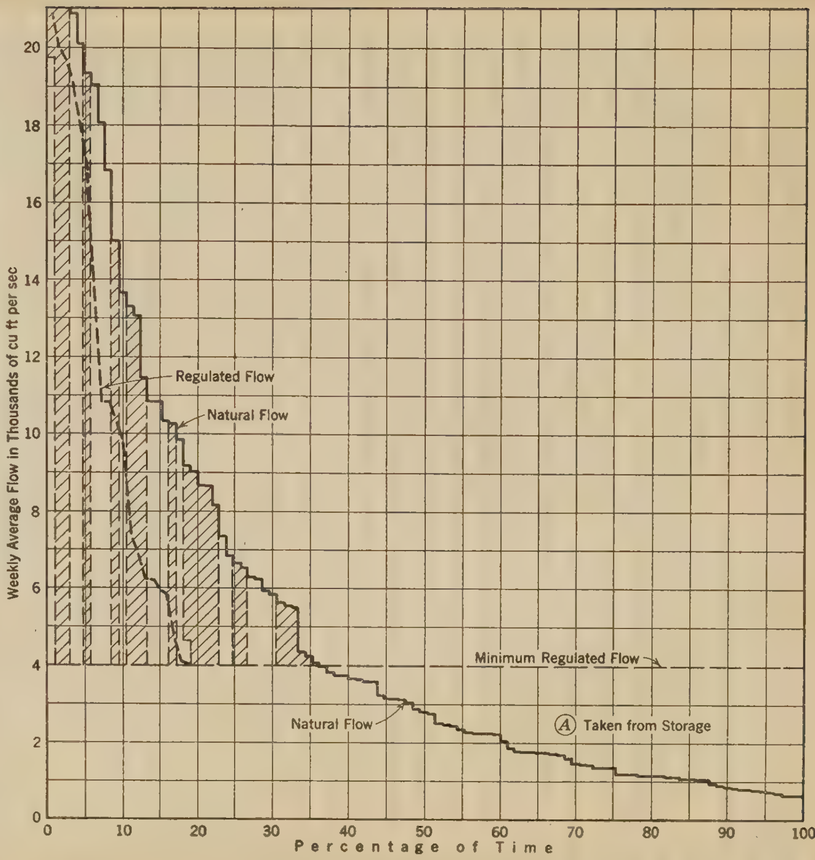

FIG. 7.—DURATION CURVES OF NATURAL AND REGULATED FLOWS, DELAWARE RIVER, PORT JERVIS, N. Y. (WEEKLY FLOW FROM MARCH 29, 1923, TO MARCH 28, 1925, INCLUSIVE).

maintain a constant rate of stream flow (4 000 cu ft per sec) during all periods of low natural run-off, it being assumed that the volume of storage available is sufficient for this purpose. When the natural stream flow is less than the minimum regulated flow, the non-regulated flow is supplemented by enough water taken from the storage reservoir to make the total daily flow equal to the regulated flow. The water taken from the reservoir for this purpose must be replaced during those periods when the natural flow is greater than the minimum regulated flow. This portion of the stream flow retained in the reservoir is indicated in Fig. 7 by the shaded areas.

The duration curve of regulated flows is obtained by re-arranging the corrected natural flows in their order of magnitude, as shown by the dotted line in Fig. 7. The section of the diagram to the left of this dotted line and above the minimum regulated flow line represents that part of the natural stream flow that would be wasted if the capacity of the power plant were only equal to the minimum regulated flow. It is equal to the sum of the unshaded areas above the minimum regulated flow line. The area between the regulated and natural flow curves, above the minimum regulated flow line, is equal to the sum of the shaded areas, and is also equal to the area between the minimum regulated flow line and that part of the natural flow curve which lies below it (marked *A* in Fig. 7).

The relation between the regulated flow curve and the volume of storage required is not shown by the duration curve, but may be determined conveniently from the mass curve, as explained in various textbooks. In a long-continued dry period, there may be one or more short periods with surplus flow, when the run-off is greater than the regulated draft, and the reservoir depletion is partly restored. In the following period of low run-off, the regulated flow is maintained for a time by drawing on this replaced storage. Taking the entire period of record, the summation of all the deficiencies is represented in Fig. 7 by the entire area marked *A*; but in determining the total volume of storage required, the intermediate surplus flows act to balance some of the deficiencies, and the required storage volume will be less than that shown by Area *A*.

Evidently, if the duration curve is made up from a record extending only from a peak of the mass curve to the next subsequent point of maximum reservoir draw-down, the required storage is equal to the sum of the deficiencies minus the sum of the surpluses for that period of record. In this case, the area between the minimum regulated flow line and the part of the curve above it is always smaller than the area between the minimum regulated flow line and the part of the curve lying below; and the storage required for the limited period of record will be equal to the difference between these two areas.[5]

In general, however, it is not possible to use the duration curve of natural flow for computing the storage capacity required to maintain an assumed regulated flow. The mass curve is a more convenient method for obtaining this storage capacity, and even in the special case noted herein the mass curve is necessary for the preparation of the duration curve.

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[5] "Application of Duration Curves to Hydro-Electric Studies," by G. H. Hickox and G. O. Wessenauer, Juniors, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 98 (1932), p. 1276.

Pondage.—A distinction must be made between storage and pondage. Storage is the reservoir capacity used to supplement deficient stream flow over a considerable length of time. Pondage is the volume of water that can be drawn upon at the power house to increase the available stream flow during certain hours of the day. This volume is replaced during the hours when the power load does not require so much water. Storage, only, is considered in determining the regulated flow. The pondage is replaced every twenty-four hours, and is not available for supplementing the natural stream flow during dry periods.

As a special case, part of the pondage may be used for "week-end regulation." The flow of the stream is retained in the pond from Saturday afternoon until Monday morning, and then released on the remaining week days to augment the regulated flow during the hours of maximum load. The effect of pondage on power output is explained in this paper under the heading, "Application to Hydro-Electric Power Studies."

THE DURATION-AREA CURVE

The duration-area curve is a useful device for studies with the duration curve. As far as the writer knows, it was first used by the late E. W. Maloney, M. Am. Soc. C. E. It may be defined as the curve showing the area within the duration curve and beneath a horizontal line having any given ordinate. When the %-of-time basis is used for plotting the curve, such an area also represents the average of the ordinates included in that part of the diagram. Hence, the duration-area curve also shows the average stream flow available below a given draft.

The duration-area curve is illustrated in Fig. 8(*a*). It is conveniently plotted at the left of the sheet, with the scale of ordinates the same as for the duration curve, and abscissas showing the area under the duration curve, or the average ordinate under that curve. (The scale of abscissas is at the top of the diagram.) Thus, in Fig. 8(*a*) the area, *ONKBC,* is represented on the duration-area curve by the abscissa, *NG*; and the average ordinate of this area is 2 500 cu ft per sec.

If the same linear scale is used for the duration-area abscissas as for the duration-curve ordinates, the former curve will intersect the axis of abscissas at an angle of 45 degrees. If the duration curve intersects the 100%-of-time line at an ordinate, y, the duration-area curve will be tangent to this 45° line at the same ordinate value. The duration-area curve will be tangent to a vertical line at a point the abscissa of which is equal to the mean run-off of the entire record, and the ordinate of which is equal to the maximum rate of stream flow in the record.

The duration-area curve is the integral of the duration curve if the stream-flow axis is taken as the base of the duration curve. The slope of the duration-area curve with respect to the stream-flow axis is equal to 0.01 times the %-of time at the corresponding point on the duration curve, if the scales are not distorted.

Just as the duration curve is much more regular than the frequency curve, so also the duration-area curve is much smoother than the duration curve from which it is derived. It can be plotted with considerable accuracy, therefore, from a relatively small number of computed points. In computing the duration-area curve, the areas under the duration curve can be measured with a planimeter, or the original data may be used.

Approximate Regulated Flow Curve

The actual duration curve of regulated flow can only be determined by a detailed study of the original stream-flow record (perhaps with the assistance of the mass curve) and the construction of a revised record of flow after regulation. This is often a tedious process. An approximate method of constructing such a regulated flow curve has been devised by Mr. E. Laurence Burnett.

This method, as applied to the case of regulation for a constant minimum draft, is illustrated in Fig. 8(*a*). The minimum regulated draft assumed for this example is 2 500 cu ft per sec. The point at which the duration-area curve reaches an abscissa of 2 500 cu ft per sec is noted, as at *G*. The horizontal line, *NG*, is drawn, intersecting the natural flow duration curve at *K*. From the properties of the duration-area curve, the average ordinate of the area, *ONKBC*, is equal to 2 500 cu ft per sec; that is, to the minimum regulated flow, *OP*. Furthermore, the area, *ONKBC*, is equal to the rectangular area, *OPDC*; hence, Area *PNKB* equals Area *BDC*. In other words, if all the surplus flows represented by the area, *PNKB*, are assumed to be stored in the reservoir, they will be sufficient to maintain the regulated draft during all periods of deficient natural flow.

Under this assumed operation, the regulated flow curve would follow the horizontal line, *DM*, as far the %-of-time corresponding to the point, *K*. The remainder of the curve would be found by shifting all that part of the natural-flow curve above *K* vertically downward by the distance, *KM*. The resulting duration curve of "approximate regulated flow" would be the dotted line, *SM*, which is at a constant distance below *AK*.

As a test of the accuracy of this method, the "actual regulated flow" curve is also plotted, as shown by the dashed line. The approximate curve is in fairly close agreement with the actual curve. Fig. 8(*b*) shows another example of this method, in which the approximate curve shows an even closer agreement with the actual curve.

In making computations with the duration curve of regulated flow, it is convenient to have a corresponding duration-area curve of regulated flow. When the approximate method of constructing the regulated flow curve is used, the proper duration-area curve is readily constructed by the following method: The area curve will follow the 45° line, *OT* (Fig. 8(*a*)) up to the ordinate equal to the minimum regulated flow. Beyond this point, it will follow a curve, *TU*, parallel to the original area curve, *GH*, and at the constant distance, *GT*, below the original curve.

In all studies of power output, the regulated-flow duration curve should be used when storage capacity is available for regulating the stream flow. If

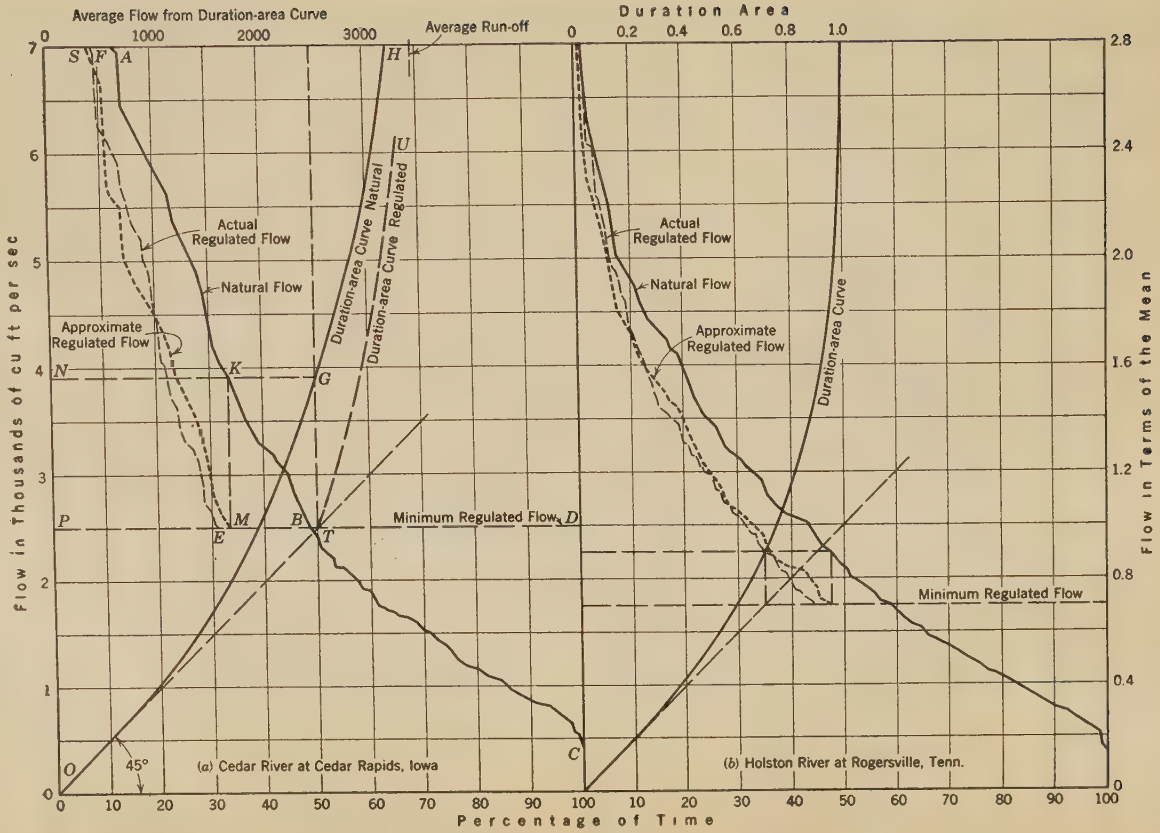

FIG. 8.—REGULATED FLOW-DURATION CURVES, MONTHLY RECORDS: (*a*) CEDAR RIVER, CEDAR RAPIDS, IOWA, 1903 TO 1920; (*b*) HOLSTON RIVER, ROGERSVILLE, TENN., 1904 TO 1925.

no storage is available, the natural-flow duration curve should be used. In either case, the proper duration-area curve will be needed, corresponding to the flow-duration curve.

APPLICATION OF THE DURATION CURVE TO HYDRO-ELECTRIC POWER STUDIES

The kilowatt-hour output of a hydro-electric plant is equal to $\frac{0.746\,F\,T\,H\,E}{8.8}$, in which, F is the flow through the turbine, in cubic feet per second; T, the length of time, in hours, during which F is available; H, the net operating head, in feet; and E, the over-all efficiency.

Factors H and E are primarily physical characteristics of the plant, which vary slightly according to load requirements, but may be considered as fairly constant. (If the plant is directly connected to a large storage reservoir, H may have considerable variation.) In the following example, H and E are assumed to remain constant; but F and T depend on the method of water use. The power produced depends closely on the total volume of water that passes through the wheel, or on $F\,T$.

Example

A plant is assumed, consisting of one unit with a maximum water capacity of 4 000 cu ft per sec; an average net operating head of 76.0 ft; and an over-all efficiency of 80.0%; that is:

$$\text{Output} = \frac{76.0 \times 0.80 \times 0.746\,F\,T}{8.8} = 5.1558\,F\,T \text{ kw-hr.}$$

Operating Schedule.—The peak load occurs 10 hr per day, 5½ days per week (= 55 hr per week); and the average peak demand equals 85% of the installed capacity, or 4 000 × 0.85 = 3 400 cu ft per sec. The off-peak load occurs 14 hr per day, 5½ days per week (= 77 hr per week); and the average demand equals 45% of the installed capacity, or 1 800 cu ft per sec. The week-end load occurs 24 hr per day, 1½ days per week (= 36 hr per week); and the average demand equals 40% of the installed capacity, or 1 600 cu ft per sec.

Water Control.—No annual storage regulation is provided, so that this is a "run-of-the-stream plant." The 24-hr natural stream flow is available, as shown by the duration curve in Fig. 9. At the plant there is a pond with a capacity of 50 400 000 cu ft, which is used to hold back the river flow during off-peak hours and release this stored water to increase the available flow during peak hours, up to the capacity of the turbine. The maximum river flow that can be held back entirely in 14 hr is $\frac{50\,400\,000}{14 \times 3\,600} = 1\,000$ cu ft per sec. In addition to the daily pondage, there is a small reservoir that can withhold natural flows equal to or less than 1 100 cu ft per sec during the 36 hr of the week-end, for release during the 55 hr of peak demand.

The daily and week-end pondage are operated independently of each other. The cycle of the daily pondage is completed every 24 hr for 5½ days per week, while the cycle of the week-end pondage requires a full week. The peak-power output is produced by the use of the natural stream flow available during the peak hours, plus the additional flow supplied by the daily and week-end pondage.

The duration curve in Fig. 9 shows the distribution of 24-hr average flows throughout a typical year. The same curve may also be considered to indicate the distribution of weekly average flows during the year, without appreciable error, so that any point on the curve represents the average flow of an entire week. If the natural flow, as shown by the duration curve, is Q cu ft per sec, the week-end pondage can hold back 36 Q cfs-hr when Q is less than 1 100, which will increase the peak use by $\frac{36\,Q}{55}$ cu ft per sec. The daily pondage will increase peak use by $\frac{14\,Q}{10}$, when Q is less than 1 000 cu ft per sec. The maximum benefit from daily pondage is $\frac{14 \times 1\,000}{10}$ $= 1\,400$ cu ft per sec, and the maximum benefit from week-end pondage is $\frac{1\,100 \times 36}{55} = 720$ cu ft per sec.

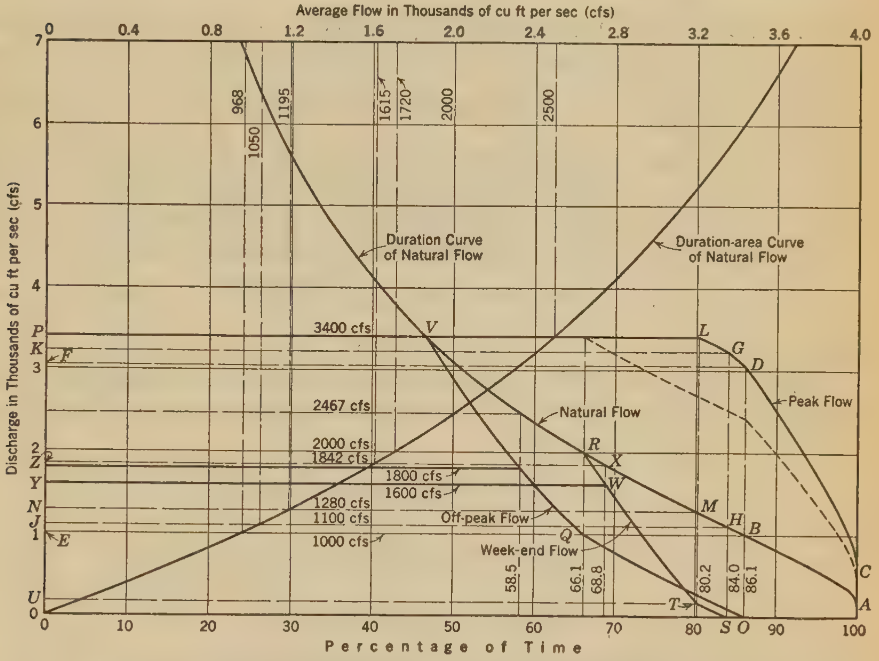

FIG. 9.—EXAMPLE ILLUSTRATING OPERATION OF A HYDRO-ELECTRIC PLANT.

Off-peak and week-end power will only be produced when the natural flow is greater than is required to refill the ponds during off-peak or week-end periods; that is, when $Q > 1\,000$ cu ft per sec for off-peak power, and when $Q > 1\,100$ cu ft per sec for week-end power.

Average Annual Power Output.—When the natural flow is so small that the daily pondage of 50 400 000 cu ft is not fully used ($Q < 1\ 000$ cu ft per sec), the peak power is produced by a flow of $Q + \frac{14\,Q}{10} + \frac{36\,Q}{55} = \frac{168\,Q}{55}$, in cu ft per sec. This is shown in Fig. 9 by the curve, *C-D*, the ordinates of which are $\frac{168}{55}$ times the corresponding ordinates of the natural flow curve, *A-B*. The maximum benefit from daily pondage is 1 400 cu ft per sec, when $Q = 1\ 000$ (at *B*). From *D* to *G*, the peak flow available is $Q + 1\ 400 + \frac{36\,Q}{55} = \frac{91\,Q}{55} + 1\ 400$. At *G*, when $Q = 1\ 100$ cu ft per sec, the maximum benefit from week-end pondage is reached. From *G* to *L*, the peak flow equals $Q + 1\,400 + 720 = Q + 2\,120$, and both the daily and week-end pondages are used at full capacity. Full peak load is supplied when $Q + 2\ 120 = 3\ 400$, or when $Q = 1\ 280$ (at *M*). The duration curve of peak flow, then, is the line, *CDGLP*, for which 100%-of-time $= \frac{55 \times 365}{7} = 2\ 868$ hr of peak operation per year.

The average peak flow is represented by the total area under the curve, *CDGLP* (Fig. 9). This may be obtained from the area under the natural flow curve, *ABHMN*, as follows:

$$\text{Area } CDF = \text{Area } ABE \times 168 \times \frac{1}{55}$$

$$\text{Area } FDGK = \text{Area } EBHJ \times 91 \times \frac{1}{55}$$

$$\text{Area } KGLP = \text{Area } JHMN$$

The areas (in cubic feet per second) under the natural flow curve may be obtained from the duration-area curve. Thus,

$$\begin{aligned}
\text{Area } CDF &= 968 \times 168 \times \frac{1}{55} \ \ldots\ldots\ldots &= 2\ 956.8\\
\text{Area } FDGK &= (1\ 050 - 968) \times 91 \times \frac{1}{55} &= 135.7\\
\text{Area } KGLP &= 1\ 195 - 1\ 050 \ \ldots\ldots\ldots\ldots &= 145.0\\
&\text{Total} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots &= 3\ 237.5
\end{aligned}$$

The average annual peak output equals $3\ 237.5 \times 2\ 868 \times 5.1558 = 47\ 870\ 000$ kw-hr.

Off-peak power is produced when $Q > 1\ 000$ cu ft per sec (at *B*). When the pond is fully emptied each day, 1 000 cu ft per sec must be deducted from the natural flow during off-peak hours to refill the pond, and the flow available for off-peak power will be $Q - 1\ 000$. When Q becomes sufficiently great, the daily pondage will not be fully used. This will occur when the natural flow during peak hours plus $1\ 000 \times \frac{14}{10}$ is greater than peak load, or 3 400; or when $Q + 1\ 400 > 3\ 400$, or $Q > 2\ 000$ cu ft per sec. Hence, when $1\ 000 < Q < 2\ 000$, off-peak flow $= Q - 1\ 000$. This is represented by the curve OQ. When $2\ 000 < Q < 3\ 400$, available off-peak flow will equal $\frac{Q \times 24 - 3\ 400 \times 10}{14}$, or $\frac{Q \times 24}{14} - 2\ 429$. This is shown by the curve, QV; but the average limit of off-peak use is 1 800 cu ft per sec, corresponding to $Q = 2\ 467$ cu ft per sec.

The total area (in cubic feet per second) under the off-peak flow curve, OQV, below 1 800 cu ft per sec, is obtained from the natural flow curve, as follows:

$$\text{Area } OQE = \text{Area } EBR2 = 1\ 720 - 968 \ldots\ldots\ldots = \quad 752$$

$$\text{Area } EQVP \text{ (below 1 800 cu ft per sec)}$$

$$= \frac{(2\ 000 - 1\ 720)\ 24}{14} \ldots\ldots\ldots\ldots\ldots\ldots = \quad \underline{480}$$

$$\text{Total} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = 1\ 232$$

In this case, 100%-of-time $= \frac{77 \times 365}{7} = 4\ 015$ hr per yr of off-peak operation. The average annual off-peak power output is, $1\ 232 \times 4\ 015 \times 5.1558 = 25\ 500\ 000$ kw-hr.

Under the assumed method of operation, no week-end power can be produced when $Q < 1\ 100$ cu ft per sec. Water for week-end power will equal $Q - 1\ 100$, as indicated by the curve, ST, when $1\ 100 < Q < 1\ 280$. For $Q > 1\ 280$, full peak load is supplied, and the week-end pondage will not be fully used. Week-end flow will then equal,

$$\frac{Q \times 168 - 3\ 400 \times 55 - (Q - 1\ 000)\ 77}{36} = \frac{Q \times 91}{36} - 3\ 056$$

as shown by the curve, TR. At R ($Q = 2\ 000$), the daily pondage is sufficient to carry full peak by itself, and week-end pondage becomes no longer necessary. Above $Q = 2\ 000$, the week-end flow $= Q$, and is represented by the curve, RV; but the average limit of week-end use is 1 600 cu ft per sec, and the curve of flows available for week-end power is $STWY$, for which 100% of-time $= \frac{36 \times 365}{7} = 1\ 877$ hr per yr. The value of Q corresponding to the point, W, is 1 842 cu ft per sec, as shown at X.

The areas (in cubic feet per second), under the week-end curve are obtained from the natural flow curve, as follows:

$$\text{Area } STU = \text{Area } JHMN = 1\ 195 - 1\ 050 \ldots = \quad 145.0$$

$$\text{Area } UTWY = \text{Area } NMXZ \times \frac{91}{36}$$

$$= (1\ 615 - 1\ 195)\frac{91}{36} \ldots\ldots\ldots\ldots = \underline{1\ 061.7}$$

$$\text{Total} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = 1\ 206.7$$

The average annual week-end power output equals 1 206.7 × 1 877 × 5.1558 = 11 680 000 kw-hr.

Waste Power.—Since off-peak and week-end loads are less than peak load, there is a waste of possible power within the limit of average peak use (3 400 cu ft per sec). The off-peak waste flow is represented by the area under the line, *QVP*, and above 1 800 cu ft per sec. This area equals $(2\,500 - 2\,000)\frac{24}{14} = 857$ cu ft per sec. Off-peak waste power = 857 × 4 015 × 5.1558 = 17 780 000 kw-hr.

Week-end waste flow (in cubic feet per second) is represented by the area, *YWRVP*, which is found as follows:

$$\begin{aligned} \text{Area } YWR2 &= \text{Area } ZXR2 \times \frac{91}{36} \\ &= (1\,720 - 1\,615) \times \frac{91}{36} \ldots\ldots\ldots\ldots = 266 \\ \text{Area } 2RVP &= 2\,500 - 1\,720 \ldots\ldots\ldots\ldots\ldots\ldots = 780 \\ \text{Total} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots &= 1\,046 \end{aligned}$$

Week-end waste power equals 1 046 × 1 877 × 5.1558 = 10 120 000 kw-hr.

Summary of Annual Power Output.—The total annual power output is as follows:

Class of power	Output, in million kilowatt-hours	Waste, in million kilowatt-hours
Peak power	48.87	0
Off-peak power	25.50	17.78
Week-end power	11.68	10.12
Total	85.05	27.90

The total power possible within the average peak demand (3 400 cu ft per sec) equals 85.05 + 27.90 = 112 950 000 kw-hr. As a check on the arithmetical work, the average natural flow below 3 400 cu ft per sec = 2 500 cu ft per sec, and the total possible output equals 2 500 × 8 760 × 5.1558 = 112 900 000 kw-hr.

The foregoing example is presented in order to show certain applications of the duration curve to hydro-electric problems. It is intended to be illustrative only, and does not by any means show all the possibilities of the method.

HISTORICAL

An attempt has been made to determine the origin of the duration-curve method, but without definite results. Apparently, it has been developed by many engineers, and its conception cannot be credited to any one man, as in the case of the mass curve. In this connection, the following statements[6] by the late John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., are of interest:

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[6] From a letter dated December 1, 1930.

> "The first printed article that I ever saw on duration curves was about forty or fifty years ago by Clemens Herschel, but I cannot remember the name of the publication. Somewhere about fifty years ago I had a hand in making a duration curve on a large scale, showing the number of days the Merrimac River at Lawrence had stood above a given height, or a given quantity. This diagram, about 30 inches high by 10 feet long, hung in a prominent place on our office wall and was frequently studied and brought up to date year by year. Probably it hangs there yet with the additions of the past half century. This was made up under the supervision of my chief, Hiram F. Mills."

There are numerous examples of duration curves in various publications subsequent to 1900. The earliest of these is in Mr. Freeman's "Report Upon New York's Water Supply," published in 1900 (page 403). In the 1903 Report of the Commission on Additional Water Supply for the City of New York, the late George C. Whipple, M. Am. Soc. C. E., presented duration curves of the daily flow of the Croton River (Appendix VI, page 486). In the Third Annual Report of the New York State Water Supply Commission, February 1, 1908, Mr. Freeman shows daily and monthly duration curves of the run-off of the Hudson River.

In all these examples, the data are plotted "in order of magnitude," but no use is made of the %-of-time method. This method, however, is used by Walter McCulloh, M. Am. Soc. C. E., in the Sixth Annual Report of the New York State Water Supply Commission, of January 31, 1911, in his report as Consulting Engineer dated December 31, 1910 (pages 119–122). In regard to this report, E. H. Sargent, M. Am. Soc. C. E., writes:

> "It is my recollection that this type of curve was original with us, although it may very well be that others entirely independently may have used this sort of curve prior to that time."

The "Introduction to the Theory of Statistics," by G. U. Yule, published in 1910, also gives examples of duration curves which are referred to as "percentile" or "integrated frequency" curves. These are plotted on the %-of-time basis. Since 1910, the %-of-time method of plotting seems to have been in quite general use.

Reference has already been made to Mr. Hazen's paper[4] on "Storage to Be Provided in Impounding Reservoirs." In this paper, the duration-curve method is used with particular regard to its probability features, and a special probability scale is developed for plotting the data. This application of the duration-curve method seems to have been entirely original with Mr. Hazen.[7]

ACKNOWLEDGMENT

The writer wishes to acknowledge the valuable assistance rendered by Mr. E. Laurence Burnett in the preparation of the numerical example included in this paper, and for numerous criticisms and suggestions.

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[7] See, also, "Flood Flows," by the late Allen Hazen, M. Am. Soc. C. E., 1930.

DISCUSSION

RICHARD PFAEHLER,[8] M. AM. SOC. C. E. (by letter).—The application of the duration curve to hydraulic problems has been clearly presented in this paper. The engineer who is called on to make a report on the feasibility of an hydro-electric, or a water supply, project must devote a considerable part of his allotted time to the collection and consequent analysis of all available stream-flow data with reference to the project; and then he must present his conclusions in such form as will make them easily understood by his client. To this end the graphic method, including hydrographs, storage curves, mass curves, and flow-duration curves, as described by the author, is often used to supplement any analytical data and tabulations.

In 1908 and 1909 the writer made a stream-flow study of the principal streams in North Carolina and South Carolina. At that time (which coincided with the beginning of the intensive development of the cotton-mill industry in the South), it was customary to develop the "run of the river," and the general rule was that a power installation corresponding to an eight months' flow was practical and economical. This, of course, suggested the preparation of flow-duration records with the use of flow-duration curves, which were usually plotted with the abscissa giving the percentage of time that the flow is equal to, or more than, any given rate. Due to the great irregularity in flow of these streams only the daily duration curves were used.

As pointed out by Mr. Foster, it is sometimes necessary to use monthly duration curves, due to the meagerness of the available stream-flow data, or due to the limited time allowed for the study. In many reports reviewed by the writer the monthly curves were used as a basis in arriving at the estimated power output, and in checking the figures furnished in such a report by the use of the daily stream-flow records, the results thus obtained sometimes differed as much as 35% to the disadvantage of the project. This last-named figure refers to rivers of the flashy type. Now, it is often quite true that the hydraulic engineer is not in a position to spend the time required for compiling, tabulating, and plotting the daily values. If this is the case the engineer should be aware of the probable percentage of error introduced by the monthly curves and should make proper allowance in his final estimates and conclusions.

It is aptly stated by the author that the duration curve represents average annual conditions for a number of years or, in other words, that it is a curve for a typical year. In this connection it is suggested that a distinction be made between a "typical-flow duration curve" and a "low-flow duration curve." The latter is meant to cover years of extreme low-water flow only, such as occurred, for instance, in the Southern States in 1904 and 1925. By this means a clear picture is presented of what may be expected in such periods of stress. This applies, of course, to both the natural and the regulated flow curve.

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[8] Design and Cons. Engr., Southern Power Co. and Allied Interests, Charlotte, N. C.

A few words on the use of the duration curve of regulated flow: As it is known, the magnitude of the minimum regulated flow depends on the available useful storage of the reservoir for the project under consideration. There is no doubt that by means of the mass curve, or by some method of tabulation of stream-flow records, the minimum regulated discharge can be determined conveniently for any year or critical season, provided the records of natural flow cover a sufficiently long period. With this information at hand, the engineer can prepare a schedule of regulated flow which, in his opinion, is best suited to the project. The writer is inclined to think that the duration curve of regulated flow will introduce difficulties when it is applied to complicated stream-flow and reservoir studies; but, for the "run-of-river" projects, the use of the duration curve is to be highly recommended.

EDWARD H. SARGENT,[9] M. AM. SOC. C. E. (by letter).—The author is to be commended for explaining in simple terms the preparation and analysis of duration curves.

The writer has written a description elsewhere[10] of the method of preparing duration curves, including about forty curves of the principal streams in New York State. Several of these curves were computed on a daily as well as on a monthly basis similar to that indicated by Mr. Foster.

The duration-area curve described by the author is a most helpful device A somewhat similar method was used by the writer[11] to obtain a so-called "energy curve" by planimetering the area within the duration curve and below a given flow or capacity line, depending on whether the ordinate was in terms of flow or power.

Mr. Foster's illustration of the application of the duration curve to hydro-electric problems is well done. However, it has been the writer's experience in analyzing the output of hydro-electric plants that the power indicated to be available by duration curves should be discounted considerably unless the output is fed into a large system capable at all times of absorbing the capacity of the plant.

C. R. PETTIS,[12] M. AM. SOC. C. E. (by letter).—A beautiful and valuable method is introduced in this paper, which can be used in the solution of certain engineering problems. Every engineering method that is assumed to be of value, possesses certain advantages. It likewise has certain limitations, and its full value cannot be realized unless the limitations are thoroughly understood.

In presenting a new method to the Engineering Profession it is especially desirable that the limitations be clearly stated; otherwise, certain engineers, who are not entirely familiar with the subject-matter, will be led into error in its application. For example, it is important to emphasize the author's explanation (see "Plotting in Terms of Mean Flow") that the run-off record at a point, *A*, on the stream may be used to represent conditions at some other point, *B*, on that stream for which there is no available record, by plotting in terms of the mean run-off. This procedure, as Mr. Foster explains, involves the assumption that the run-off for simultaneous periods at the two points is proportional to the long-term average run-off at these points. Such an assumption is not necessarily true at all times; but it may form the only feasible basis for an estimate of stream flow at a point where no records are available. In other words, the author states clearly that the method has limitations and that the limitations are more or less serious; but he does not give a scientific analysis of the limitations.

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[9] Chf. Engr., Hudson River Regulating Dist., Albany, N. Y.

[10] Annual Rept., New York State Conservation Comm., 1919, p. 463 *et seq.*

[11] An illustration of such a curve will be found on Pl. 12, Annual Rept., New York State Water Supply Comm., 1911.

[12] Lt.-Col., Corps of Engrs., U. S. Army, Fort Hayes, Columbus, Ohio.

Under "Graphical Methods for Studying Stream Flow," the author indicates that duration curves may be divided into two classes. For convenience these may be called: (1) Flood duration curves; and (2) power duration curves. While Mr. Foster limits his paper to Class (2), this discussion will indicate briefly the methods by which an analysis can be made of Class (1) in the belief that a somewhat similar, fairly exact, analysis can be made of power duration curves.

Due to the need for brevity, this discussion is neither full nor complete; it is not intended as a presentation of the subject, which would require a discussion of limitations. The principal limitation is that the flow must not be controlled to any great extent by lakes, dams, or otherwise, and the discussion is based primarily on humid conditions. Certain modifications are necessary in applying the method to semi-arid and arid regions.

In 1924, the author presented[13] a mathematical method to be used in connection with flood (or other) duration curves. That paper is the foundation of much of the work that is being done to-day, and it marked a turning point in the trend of thought on the subject. It did not define the limitations of the method, however, and, as a result, certain engineers, with a mathematical inclination, have applied the method to 20-yr records, and have freely predicted the 1 000-yr flood, or even the 10 000-yr flood from them. Some engineers apparently thought that this was going too far, and, consequently, have condemned the method without qualification.

There is some evidence, supported by practical considerations, which indicates that the duration curve is governed by slightly different laws for the section below 100 yr, and the section above 100 yr, which includes the rarer floods. A too free extension of curves, based on short records, above 100 yr, is questionable.

There are four semi-independent methods of obtaining the probable 100-yr flood at any given station. The Foster probability method, which requires a river record, is one of these methods. The probable 100-yr flood, Q_{100}, can be obtained for any station in the United States, with or without a record, with a fairly high degree of accuracy. Practically all such determinations can be made with a probable error considerably less than 10 per cent. If such determinations are used to check the accuracy of the Foster probability determinations based on 20-yr records, it will be found that they have a probable error of about 25 per cent.

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[13] "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

Furthermore, about four out of five of such determinations will give values that are too low, this being due to the nature of the records, and the relative rarity of larger floods. In other words, the blind application of the Foster method to 20-yr records will give results that are erroneous and misleading. The fault does not lie in the method, but in the inadequacy of the data, which are used as a "sample."

The writer's width formula[14] for the probable 100-yr flood is:

$$Q_{100} = C\,P\,W^{1.25} \qquad (1)$$

in which, P is a rainfall index (the probable 100-yr rainfall), and W is the average width of the drainage area, in miles. It is desirable, in general, that a flood study should cover a group of stations, in the same general section of country, so that the study can be placed on a statistical basis. The width formula meets every statistical requirement, including close agreement with determinations by the Foster method when the records are more than for 40 yr. For the purpose of checking results, all river records in the section under study, and all records that are long enough to be of any statistical value, should be subjected to the Foster method, the mean formula, and the logarithmic method. When thus used and checked, the width formula is both simple and accurate.

In giving a value for Q_{100}, Equation (1) locates one point on the duration curve for the station in question. Most short-time records, such as 20-yr records, will give duration curves that are not truly representative, and that pass below Q_{100} as plotted.

The mean maximum annual flood, or the probable 1-yr flood, Q_1, may be obtained from a river record, or may be computed by the formula,

$$Q_1 = C\,P\,W^{1.05}\,L^{0.5} \qquad (2)$$

in which, L is the length of the river, in miles. This gives the mean flood with reference to which other values on the duration curve may be plotted.

The probable 100-yr flood in terms of the mean is,

$$\frac{Q_{100}}{Q_1} = C\,W^{0.2}\,L^{-0.5} \qquad (3)$$

Now Q_1 fixes the mean line of the duration curve; Q_{100} gives an upper point on the curve; and, $\frac{Q_{100}}{Q_1}$ fixes a definite relation between the coefficient of variation and the coefficient of skew. The curve is now fixed within fairly narrow limits. It is fully fixed if a value is assumed for either the coefficient of variation or the coefficient of skew.

The mean formula for the 100-yr flood,

$$Q_{100} = C\,Q_1\,W^{0.2}\,L^{-0.5} \qquad (4)$$

will give a probable error of less than 10% even with a 20-yr record, which generally gives a fairly good value of the mean, and it is believed a fairly good value of the coefficient of variation. The coefficient of skew is the principal element of uncertainty of the Foster method with short-time records.

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[14] "The Probable 100-Year Flood," by C. R. Pettis, M. Am. Soc. C. E. (Published privately.)

The duration curve as plotted gives values that will probably be equalled or exceeded in different frequency periods. The writer prefers to use the probable maximum flood that will occur in a given frequency period, F.

The following approximate relation can be used to convert from one to the other: The probable maximum flood in a given period, F, is equal to the flood that will probably be equalled or exceeded in a period of 2.5 F yr. A consideration of this relationship suggested the logarithmic method which is indicated by the formula,

$$Q_{100} = Q_1 \, (1 + 6.6 \, K) \quad \ldots\ldots\ldots\ldots (5)$$

in which, K is obtained from the relation,

$$K = \frac{Q_{2F} - Q_F}{Q_1} \quad \ldots\ldots\ldots\ldots (6)$$

An average value of K can be obtained from a river record, by using values of F of 1, 2, 4, 8, etc. This method is much simpler to apply than the Foster method. In general, it seems to be almost, but not quite, as accurate as that method.

All formulas given in this discussion were derived rationally, one step leading to another. They have all been checked against actual river records. Collectively, they are far more accurate than any other method of flood prediction that has come to the writer's attention. Flood prediction is approaching the point where it can be classed as an exact science, the foundation being statistical.

Specifically, it is possible to construct a synthetic flood duration curve for some river point, B, without a record, with the knowledge that the synthetic curve will probably represent conditions better than a curve transplanted from some other point, A. At points down stream there is a tendency, subject to temporary reversals, for the values of $\frac{Q_{100}}{Q_1}$ to decrease decidedly.

It is believed that the laws governing power duration curves offer an interesting field of investigation, and that the analysis will be no more complicated nor difficult than was the similar analysis of flood duration curves.

W. G. HOYT,[15] M. AM. SOC. C. E. (by letter).—One phase of the duration curve, which is of national and international interest, is not discussed by Mr. Foster. In 1909 the U. S. Geological Survey prepared the first estimates of the potential power of the United States for President Theodore Roosevelt's National Conservation Commission. In making these estimates,[16] two rates of flow were used—a so-called minimum flow obtained by averaging the two consecutive 7-day periods of lowest flow in each year, and a higher rate based on the stream flow that could be assured for six months of the year.

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[15] Hydr. Engr. (Prin.), Conservation Branch, U. S. Geological Survey, Washington, D. C.

[16] *Water Supply Paper No. 234*, U. S. Geological Survey.

These rates are fairly comparable with those still being used by the Canadian Water Power Service. About 1919, the Geological Survey undertook a compilation of world power statistics for use by President Wilson's advisers at the Paris Conference following the World War. As detailed estimates of stream flow were not available for many countries, great refinement was impossible, and where more reliable information was not available the estimates of power were based on a flow believed to be available 75% of the time.

During the period, 1910 to 1920, there was a rapid increase in the use of the duration curve among engineers, largely, it is believed, as a result of its adoption by the New York State Water Supply Commission. On account of its many apparent advantages, the duration curve was adopted by the U. S. Geological Survey about 1920 as a basis for defining rates of flow to be used in computing water-power statistics. At that time two rates of flow—namely, the flow available 90% of the time (designated $Q90$) and the flow available 50% of the time (designated $Q50$)—were adopted as Geological Survey standards.

When the Federal Power Commission was created, in 1921, the Commission adopted the $Q90$ rate of flow as a basis for computing the project horsepower capacity on which administrative charges were to be levied. At a somewhat earlier date the Wisconsin Railroad Commission adopted the $Q50$ flow for estimating the horse-power capacity of sites under license by that Commission.

At the World Power Conference in London, England, in July, 1924, reports were presented on the developed and undeveloped resources of the various countries represented. These reports used no common standards for stating the power resources of rivers, and, therefore, it was impossible to make reasonably satisfactory comparisons of the resources of the different countries. After the publication of an editorial in the *Electrical World* for October 11, 1924, entitled "Wanted—A Standard Method for Rating Rivers," the American Engineering Standards Committee (now the American Standards Association), authorized the appointment of a Sectional Committee to determine bases for computing, and units for expressing, water-power resources for statistical use and for the purpose of making comparisons between different rivers, river basins, regions, and countries.

After two years of discussion, this Sectional Committee tentatively adopted the $Q90$ and $Q50$ rates of flow and recommended that these rates be used, with the gross head and an assumption of 100% efficiency, to compute the theoretical power of rivers, in kilowatts. Consideration of these rates by the American Standards Association is being held up (1934) pending international consideration.

In 1927, an International Advisory Committee on Rating of Rivers was created by the International Electrotechnical Commission. The countries making up this International Committee included Canada, Czecho-Slovakia, France, Germany, Great Britain, Italy, Norway, Sweden, Switzerland, and the United States. The United States National Committee acted as Secretariat, with the personnel of the Sectional Committee previously mentioned acting as the United States Advisory Committee. After protracted correspondence and meetings, the International Advisory Committee, at a meeting in Stockholm, Sweden, on July 4, 1930, adopted three rates of flow—namely, the flow available 95% of the time (or approximate equivalent); the flow available 50% of the time; and the mean flow. On account of difficulties in translation, these rates have not yet (1934) been officially adopted by the International Electrotechnical Commission, the French National Committee insisting thus far on defining each rate of flow as the average of the duration curves for each year rather than the duration curve for the period as a whole. It is expected that this difficulty will be straightened out soon, and that the rates recommended at the Stockholm meeting will be officially adopted by the International Electrotechnical Commission. In the meantime, the World Power Conference has adopted as a basis for its international power statistics the rates of flow as set forth, using gross head, 100% efficiency, and $Q95$, $Q50$, and the mean flows, and expressing the theoretical capacity in kilowatts. All engineers who have attempted to reconcile estimates based on rates of flow other than the duration curve will realize the importance of a national and international standard. It is doubtful whether agreement could have been reached on the basis of any defined rates of flow other than those obtained through the use of the duration curve. Reasons justifying national and international adoption include the following: (*A*) It is in widespread use among engineers in many countries; (*B*) rates of flow based on the curve are easily defined; (*C*) defined rates of flow can be easily checked by independent computation; and (*D*) the curve presents a generalized picture of the flow and the relation of flows of various magnitude to the time covered by the record.

There is some confusion among engineers as to the relation between the duration curve and the deficiency curve. The latter may be defined as a curve the abscissa of which at any point shows the number of time units or the percentage that the flow was less than the amount shown by the ordinate of the curve at that point. A duration curve may be defined as a curve the abscissa of which at any point shows the number of time units or the percentage that the flow equalled or exceeded the amount shown by the ordinate of the curve at that point. A deficiency curve is readily convertible into a duration curve, or *vice versa,* by using the component of the time units. A duration curve may well be considered a special form of hydrograph, since it is made from exactly the same data used in the preparation of a hydrograph. It is arranged differently, however, the discharge data being plotted in order of magnitude rather than in order of occurrence. The area under a hydrograph represents the total flow regardless of whether the hourly, daily, weekly, or monthly data are plotted. Likewise, the area under a duration curve represents the total flow regardless of whether the time unit used in its preparation is the hour, day, week, or month.

The hydrograph indicates by its form the time unit that has been used. The plotted duration curve by its form does not show the time unit, and, therefore, the explanation must always indicate the time unit used. A duration curve that averages rates of flow corresponds to a hydrograph that is based on averages and is not a true duration curve any more than a hydrograph thus prepared is a true hydrograph. As the area under duration curves based on hourly or daily observations or weekly or monthly means is the same, curves based on any two of these different units will always cross each other, and the negative area will equal the positive area. For example, curves based on daily observations and monthly means will be separated at the minimum discharge end by an amount equal to the difference between the minimum daily discharge and minimum monthly mean, and in a similar manner curves will be separated at the maximum end by an amount equal to the difference between the corresponding maxima. The difference between curves based on hourly, daily, weekly, or monthly data is an indication of the magnitude of the fluctuation of the flow within the same time unit. The difference between duration curves based on daily discharge and monthly discharge, when expressed in terms of volume, will represent the amount of storage necessary to take care of variations in flow through the month over that shown by the mass curve based on monthly data.

ADOLPHO SANTOS, JR.,[17] JUN. AM. SOC. C. E. (by letter).—This presentation is lucid and concise. The author makes possible a better understanding of the meaning, limitations, and uses of duration curves. However, certain aspects of the subject will bear elaboration to advantage. Quite briefly, Mr. Foster states that duration curves may be represented by theoretical equations or curves in accordance with the methods set forth in his earlier paper.[8] He then explains in great detail the more elementary methods of constructing duration curves, but makes no further reference to the theoretical method which he himself has made so easy to apply.

Properly, he also remarks that when an attempt is made to represent the data by a continuous line, instead of by a histogram, it is tacitly assumed that there are an infinite number of items to be plotted; that is, it is assumed that the record is of infinite length; but a record of infinite length results in a smooth curve. It should follow, therefore, that when it is desired to represent the data by a continuous line, this line should not be broken or irregular, but should take the form of a continuous smooth curve. This appears to be a necessary conclusion following from the premises, subject only to the proviso that the area under any section of the resulting smooth curve must be the same as that under the corresponding section of the histogram.

For the case of stream flow the old saying, "Natura non facit saltum," still holds good. This is an additional reason why duration curves should be represented by smooth continuous lines when they are supposed to be based on records of infinite length and to represent average conditions for a stream. The writer cannot understand, for instance, the reasons why the duration curve for the Watauga River, at Butler, Tenn. (see Fig. 5), is so irregular. He does not understand the necessity for the many breaks, points of inflection, and other irregularities.

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What the hydraulic engineer usually knows is that certain observations have given certain values of stream flow. These values admittedly are merely a "sample" and do not represent exactly the entire "universe of data." It is to be expected, therefore, that there will be deviations between a duration curve representing average conditions (the universe) and the actual observed values, or sample, used as the basis of study. Conversely, a duration curve representing average conditions should not be made to follow exactly the observed values. The best use of the available data will result from the following procedure (it will be noted that this procedure is precisely that recommended in the author's earlier paper):[18]

1.—Arrange the data in order of their magnitude. If they are too numerous divide them into classes of suitable intervals; that is, group all items that lie between certain judiciously chosen limits, and treat the class as one observation of weight equal to the number of items in the group.

2.—Express the observations in terms of their general mean.

3.—Determine the plotting position of each observation on the percentage-of-time axis from $100 \dfrac{(m - 0.5\,g)}{n}$, in which m is the summation of the number of items; g, the number of items in the group; and n, the total number of items.

4.—Plot the record data, using percentage-of-time as abscissas and observations in terms of their mean as ordinates.

5.—Compute the coefficient of variation and the coefficient of skew.

6.—Select the proper type of duration curve to represent the data.

7.—Compute points for the duration curve by means of the factors in the tables given previously by Mr. Foster.[19]

8.—Plot the theoretical duration curve on the same diagram with the record data, and draw a smooth curve through the plotted theoretical points, noting how well such a curve agrees with the record data. If the agreement, in any section of the duration curve, is unsatisfactory, and if the actual data show a well marked tendency to arrange themselves in some other way than the theoretical curve, that tendency should be respected as recommended[20] by the late Allen Hazen, M. Am. Soc. C. E. The final curve as drawn for use should be that which follows the theoretical points and other conditions most closely and, at the same time, agrees with the record data in a satisfactory manner.

In taking the factors from the tables as mentioned under Item 7 it has been the writer's practice to use the coefficient of skew as computed, without any attempt at adjustment. The writer finds that the empirical adjustment factors suggested by Messrs. Foster[21] and Hazen[22] are satisfactory for flood frequency distributions, but do not seem to apply to the usual stream-flow duration curves. It is possible that if the computed coefficients of skew were adjusted in some manner or other the "goodness of fit" between the theoretical curve and actual observations might be improved. It is also possible that a curve of some type other than Pearson's Type I or Type III might afford a better fit. The writer would appreciate it if the author would express his opinion on these and similar points.

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[18] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924) p. 202.

[19] *Loc. cit.*, pp. 162, 196, and 199.

[20] "Flood Flows," by Allen Hazen, M. Am. Soc. C. E., Chapter VIII, 1930.

[21] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 175, 198, and 200.

[22] "Flood Flows," pp. 44 *et seq.*

The application of the method outlined has given excellent results in a large region of Southern Brazil for a great number of streams offering a wide range of variation of drainage areas and average monthly run-offs. The agreement between the theoretical duration curve and the plot of the actual data has been, in most cases, remarkably close. In only a few cases has it been necessary to shift or to modify the theoretical curve to improve the fit.

It is to be remarked also that the computation of the coefficients of skew and variation furnishes a definite basis of comparison for the records of different streams, or of different stations in the same stream. It makes easier also an estimation of the long-time duration curve of a stream of which only a short record, or no record at all, is available. In such cases the coefficient of skew, which appears to be normal for the region in which the stream is located, is assumed and the coefficient of variation is computed from the short record, inasmuch as it is not largely affected by the length of the series, or, in case no record is available, it is assumed also. Having the coefficient of skew and the coefficient of variation the duration curve follows easily.

JOHN D. WATSON,[23] JUN. AM. SOC. C. E. (by letter).—The author has commented on the difficulty of preparing a flow-duration curve from daily run-off records, and the relative inaccuracy of a monthly duration curve. In a study[24] of flow records on five major streams of North Carolina, the writer considered these questions in some detail.

The most accurate method of plotting a duration curve is to arrange all the items in their relative order of magnitude. When dealing with daily or weekly records covering several years, this method may be simplified to advantage by combining the items into groups having a range of about 10% of the mean flow. This may be called the "total-period method" of plotting. An easier manner of preparing the table is to arrange the run-off values for each calendar year in order of magnitude, and to average the corresponding items for all the years. This may be called the "calendar-year method." With the latter method there will be 365 points to plot for a daily record, or 52 points for a weekly record. The method is not suited for use with monthly records.

While the calendar-year method is less accurate than the total-period method, it has these advantages: (1) The tables, once prepared, can be used again without re-arrangement when additional years of record are secured and, therefore, a new up-to-date duration table can be computed with a minimum of time and effort; (2) as there will always be the same number of mean values (that is, 52 for a weekly record), the cumulative percentages-of-time for plotting will always be the same for all streams being studied on the same basis; and (3), should it be desired to plot the duration curve for any single year, its record would be already arranged in order of magnitude, and the %-of-time values would be the same as for the average year. All these advantages would appeal to those agencies, mostly governmental, which are regularly engaged in the collection of stream-flow data, and which periodically publish reports of their findings. The calendar-year method results in a curve that is higher for low flows and lower for high flows than the corresponding total-period curve, but the total area under each curve is the same.

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[23] Instr. in Civ. Eng., Univ. of North Carolina, Chapel Hill, N. C.

[24] "An Investigation of the Flow Duration Characteristics of North Carolina Streams," by Thorndike Saville, M. Am. Soc. C. E. and John D. Watson, Jun. Am. Soc. C. E., *Transactions*, Am. Geophysical Union, Fourteenth Annual Meeting, 1933, p. 406.

As to the relative accuracy of these two methods of plotting, the writer has found that, for Western North Carolina streams, the average difference in ordinates of curves plotted by the two methods is between 3 and 6%, although the percentage errors for the low flows are considerably greater. By an arbitrary correction the average error can be reduced to less than 2 per cent.[25]

The writer has recently investigated the relative accuracy of duration curves prepared from average daily, weekly, and monthly flows, for North Carolina streams, and has found the average difference between the ordinate of the weekly curve and the ordinate of a daily curve to be from 5 to 9% of the mean flow. This was nearly one-half as great as the error of the monthly with respect to the daily curve (which was found to range from 13 to 19 per cent). Because of this error, and because the weekly curve has no advantages peculiar to itself, its use is not to be recommended unless, of course, only weekly data are available.

The length of the record will also affect the accuracy of the resulting curve. Flow records of 31 to 33 years in length were analyzed to locate in each record the periods of ten consecutive years that had the highest and lowest average flows. From each of these 10-yr periods, duration curves were prepared and compared with the corresponding curve for the entire period of record. When plotted in terms of the mean flow for the corresponding period, the curves were found to be largely self-compensating. When the average flow was high, the duration curve (plotted in terms of its mean flow) was low, and *vice versa.* The conclusion was reached that a duration curve prepared from any 10-yr record of daily flows and plotted in terms of its mean is more accurate than one prepared from a 30-yr record of average monthly flows.

HOWARD L. COOK,[26] JUN. AM. SOC. C. E. (by letter).—The relation of the duration curve to the frequency curve has been well brought out by the author, and this is possibly the most important feature of the paper. Its significance should be fully grasped. Mr. Foster's paper is admirable because he has confined himself to fundamentals.

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[25] "An Investigation of the Flow Duration Characteristics of North Carolina Streams," by Thorndike Saville, M. Am. Soc. C. E., and John D. Watson, Jun. Am. Soc. C. E., *Transactions,* Am. Geophysical Union, Fourteenth Annual Meeting, 1933, p. 411.

[26] Hydr. Engr., Soil Erosion Service, Washington, D. C.

The frequency curve is used by statisticians almost to the exclusion of its integral curve (the so-called duration curve.) While the duration curve has undeniable advantages for use in several kinds of engineering analysis, it would be much better if engineers would follow the statisticians and always study first the frequency curve of their data. The duration curve is a mass curve and should be thought of as derived from the frequency curve, which is basic. An integral curve always irons out, that is, obscures, important variations easily noticeable on the curve from which it was derived. For this reason the frequency curve should be first constructed and studied and the duration curve made up from it. An engineer does not study the mass diagram of railway earthwork to familiarize himself with the characteristics of the line; he studies the profile.

It should be noted that the frequency distribution (the data, arranged in groups or classes, from which a frequency diagram or curve is plotted), is always made up for equal intervals of the variable. Therefore, when data are being tabulated for use in constructing a duration curve they should be grouped into equal intervals so that the frequency distribution, as well as the cumulative distribution for the duration curve, will be available. The construction of frequency curves is treated in all books on mathematical statistics.[27]

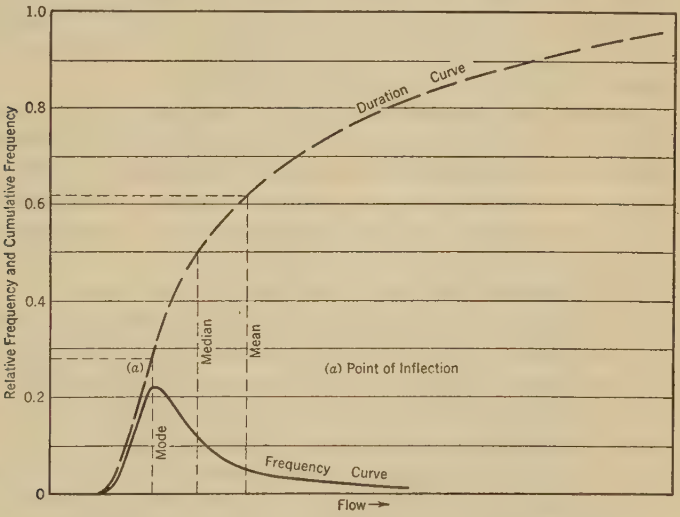

FIG. 10.—THE FREQUENCY CURVE AND ITS INTEGRAL CURVE (DIAGRAMMATIC).

An important consequence of viewing the duration curve as the integral of the frequency curve is that it leads one directly to an understanding of the proper shape for the duration curve. Fig. 10 shows a smoothed frequency curve of stream flow with its integral or mass curve plotted on the same axes. By examining the diagram it is easily seen that the point of inflection near the lower end of the duration curve occurs because there exists at that flow a maximum value of the frequency. The value of the flow at which this maximum frequency occurs is the mode. Stated otherwise, the modal flow is that which occurs most frequently.

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[27] See, for instance, "Handbook of Mathematical Statistics," H. L. Rietz, Editor-in-Chf., Houghton Mifflin Co.

It is now clear that whenever there is a modal flow greater than the minimum, there must be a point of inflection on the duration curve. Furthermore, it is well known that there is such a modal flow on all streams not intermittent in character. This follows directly from the universally known fact that the very smallest and the very largest flows occur infrequently. Consequently, there must be some intermediate flow that occurs more frequently than any other. This is the modal flow.

That the point of inflection on the duration curve has a real and important significance should be well noted. There has been a rather widespread impression among engineers that the inflection is due to errors in data and that "theoretically" the curve should not inflect. When the duration curve is viewed as the integral of the frequency curve it is perfectly obvious that this is entirely wrong. How such an erroneous proposition should have gained such wide credence is very puzzling, especially since Adolph F. Meyer, M. Am. Soc. C. E., has pointed out[28] the true significance of the inflection point and has suggested adopting the mode as the "normal."

It is true that some frequency distributions do not have a mode. Examples of these are distributions of daily rainfall in humid regions, for which the amount designated as trace, occurs most often. For such a distribution, of course, the cumulative frequency curve would not inflect. However, all discharge records of non-intermittent streams show skew distributions with modal values greater than the minimum flow.

It may be well to discuss this modal value further in connection with duration of stream flow curves. There are three important statistics, or characterizing values, that should be calculated in all analyses of stream-flow records: (*a*) The arithmetic mean or average flow; (*b*) the median flow; and (*c*) the modal flow. All three are indicated on Fig. 10. Note that, for a skew frequency curve, the median always lies between the mode and the mean.

The calculation of the average flow is always made, of course. The median flow is often noted also, it being the 50% flow on the duration curve; that is, the flow that has one-half the flows greater and one-half smaller than itself. The modal flow, however, is very rarely calculated or used, although it is suggested that, for a number of purposes, it is the most valuable of all characterizing statistics.

The modal flow is closely connected with the normal ground-water, or other form of storage flow, from a drainage basin. It provides the most sensitive single statistic available: (1) For studying the effect of changes in forest cover, etc., on stream flow; (2) for testing the consistence of stream-flow records; and (3) for comparing the natural storage characteristics of different areas. There is not space to enumerate all the advantages of this statistic in stream-flow analysis.

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[28] "Elements of Hydrology," by Adolph F. Meyer, Second Edition, p. 119, N. Y., John Wiley & Sons.

It is difficult to locate the mode correctly. The theoretical mode is the value of the variable corresponding to the maximum ordinate of a theoretical frequency curve fitted to the data. The fitting of frequency-curve equations to engineering data is entirely too ponderous a procedure for general use. For this reason it is suggested that, with care, an approximate mode may be estimated in nearly all cases of stream-flow analysis, by a method due to Czuber.[29]

The author's method of smoothing a duration curve is not entirely rational. The rational method is to smooth the frequency curve from which the duration curve is derived. Mr. Foster calls attention to the danger of using too long a time unit for a duration curve to be used in making a detailed analysis. It is not generally recognized that there is also danger of choosing too short a period. This is said with reference to the use of winter stream-flow records taken under ice conditions. In the Northern States the large amount of estimation necessitated in preparing such a record makes it impossible to obtain even a fair degree of accuracy for the mean daily flows, although the mean monthly flows may be fairly well determined. For such a record nothing is added to the results by basing computations on duration curves constructed from daily flows. Such a procedure may even prove to be misleading. The engineer is usually only justified in using mean monthly flows. In such cases two duration curves should be prepared, one on a daily basis for the summer season records, and one on a monthly, or perhaps weekly, basis for the winter period.

In connection with the history of the duration curve the following may be of interest. The cumulative frequency curve is usually called an "ogive" by the statisticians and has been used in statistical studies for many years. It came into prominence through being extensively used by Sir Francis Galton in his studies of inheritance. For this reason it is sometimes called Galton's ogive. The first published paper on the duration curve of stream flow seems to have been that of the late Clemens Herchel, Past-President and Hon. M. Am. Soc. C. E.,[30] who stated that the use of the curve[31] was suggested to him by the late Joseph P. Davis, M. Am. Soc. C. E.

C. S. Jarvis,[32] M. Am. Soc. C. E. (by letter).—The profession is indebted to Mr. Foster for his notable contribution regarding duration curves and their fields of usefulness, as well as their practical limitations. Of immediate interest is the historical account of their development and adaptation to problems related to stream flow; while the methods devised by the author for deriving new curves from the originals, to illustrate the influence of storage and regulation, have great practical value.

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[29] "Die Statistischen Forchungsmethoden," von E. Czuber; and, "Handbook of Mathematical Statistics," p. 27.

[30] "The Gauging of Streams," by Clemens Herschel, *Transactions*, Am. Soc. C. E., Vol. VII (1878), p. 236.

[31] See, Tenth Census, 1880, Water Power of United States, Pt. 1: "Report on Region Tributary to Long Island Sound," by Dwight Porter, p. 6.

[32] Prin. Hydr. Engr., U. S. Engr. Office, Washington, D. C.

In 1931, during a survey of the streams of the Delaware River System, Mr. Foster and the writer were associated on a problem that permitted a graphical solution. Fig. 11 shows the forms of duration curves for the Delaware River, before and after the proposed partial regulation incident to the diversion planned to augment the water supply for New York City. When the Court decision limited the authorized diversion to about 70% of that which was originally planned, it was evident that the departure of the proposed regulated flow curves from those showing observed discharge, would be reduced in approximately the same proportion. Moreover, it is clearly demonstrated from such diagrams that the flood peaks will be reduced, and the minimum flows increased, by such regulation attending the proposed diversion.

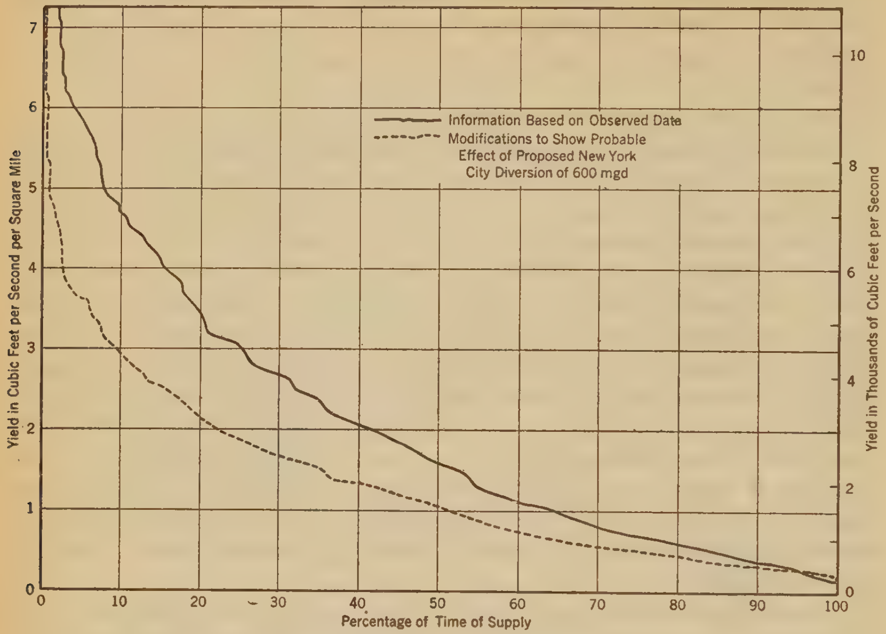

FIG. 11.—FLOW DURATION CURVES, BASED ON MEAN MONTHLY DISCHARGE OF COMBINED EAST AND WEST BRANCHES OF DELAWARE RIVER, AT HANCOCK, N. Y.

Fig. 12 portrays the duration curves for the Mississippi River at the outlet of Pokegama Reservoir, 330 miles above Minneapolis, Minn., through which are discharged the storage and the natural flow from Leech Lake and Lake Winnibigoshish as well. This is a fair measure of the difference between mean annual and mean monthly discharges as basic data on a regulated stream, and is much less than would occur with no regulation by storage. In this instance, reduction to a daily or weekly basis would alter the curve of mean monthly flow only slightly, due to the excess storage capacity, thorough regulation, and sluggish flow. In fact, it has been demonstrated by long experience that daily reports of variations in gauge heights and in discharge from these head-waters reservoirs are not so satisfactory and dependable as

10-day means. Slight wind velocities on such extensive water surfaces, a total of 440 sq miles when at full stage, may either overcome or multiply the daily draw-down of reservoirs as measured on gauges, thus making a daily record irregular and inconsistent.

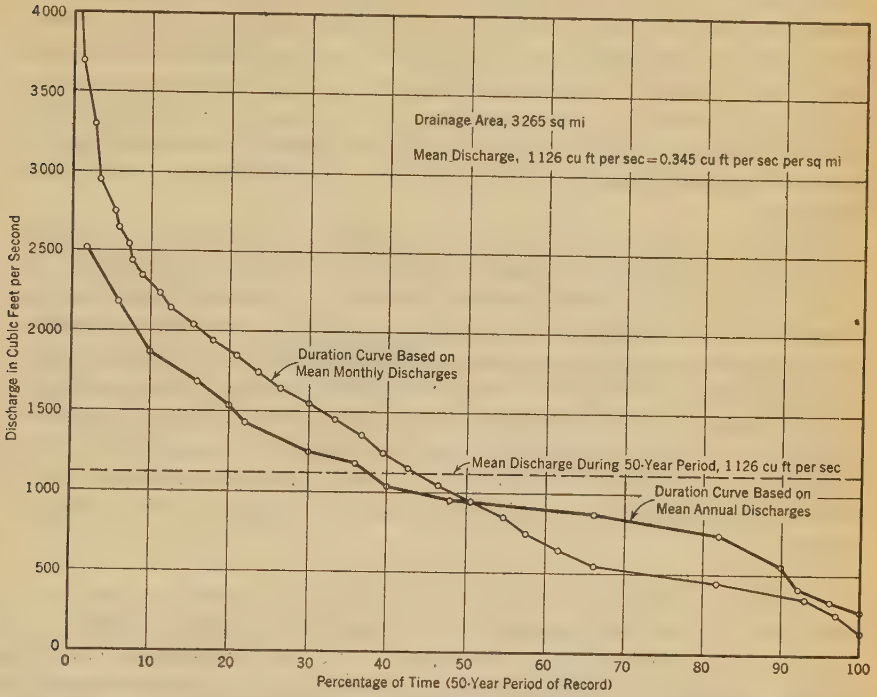

FIG. 12.—DURATION CURVES, MISSISSIPPI RIVER, AT OUTLET OF POKEGAMA RESERVOIR (COMPARING CURVE BASED ON MEAN MONTHLY AND MEAN ANNUAL DISCHARGES, RESPECTIVELY, FOR THE 50-YEAR PERIOD, 1884 TO 1933, INCLUSIVE).

DONALD H. MATTERN,[33] JUN. AM. SOC. C. E. (by letter).—The literature on the development and uses of the duration curve has certainly been enriched by this paper. The detailed explanation regarding preparation of data prior to curve plotting should assist in arranging the information in the most logical manner to secure the best results.

Despite the different papers that have explained the use of the duration curve in connection with hydro-electric plant operation, it seems that there is one fundamental objection to its widespread use: It does not present a picture of the stream flow in chronological order. In combined systems where hydro-electric power may furnish base-load power during some parts of the year and peak load the remainder of the time, it is particularly important to know the flow distribution. In either the study of a proposed development, or the operation of an existing plant in such a combined system, the steam and hydro-electric capacity must be handled in such a way that they fit most economically into the system annual load curve as well as the daily load curve. The hydrograph and the similar curve showing kilowatt-hours present this picture, whereas the duration curve cannot do so because of its construction.

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[33] Asst. Hydr. Engr., Gen. Eng. and Geology Div., Tennessee Val. Authority, Knoxville, Tenn.

Then, again, in most of the larger systems where power is developed from two or more rivers having different flow characteristics, the hydrograph will enable the engineer to operate a system more efficiently. The writer is acquainted with one system in which flow is somewhat different on the principal power streams. Total annual run-off is proportional, but the peaks have seldom occurred at the same time. The duration curve, then, would not be of value in plant operation during the period of the year when the greatest opportunity exists for savings to be effected through well-planned operation.

By use of the hydrograph it is much easier to demonstrate to executives the manner in which the output of a hydro-electric plant fits into the system; the manner in which possible changes in operating conditions are brought about by storage at one plant in one part of the year for benefit in another period; and the necessary steam-plant operation under varying conditions. Executives want to see facts presented graphically, not mathematically, although the latter may produce the same result. If use of the hydrograph type of curve is necessary to present data to the executive, is it not better to make use of it at other times as well? In other words, is not the field of the duration curve elsewhere than in connection with operating studies?

F. Knapp,[34] Esq. (by letter).—The duration-area curve is one of the finest tools the engineer possesses for solving hydro-electric problems. Together with the hydrograph and the mass curve, the duration-area curve has proved exceedingly valuable and, at the same time, is generally applicable without making limiting assumptions, especially in those complicated cases where several power stations supply the same power market. Certain properties of the duration-area curve, not mentioned by the author, lead to surprisingly simple solutions of hydro-electric problems.

The daily variable load demand of a power system changes with the seasons and, therefore, is represented by a three-dimensional diagram or "load demand mountain." In order to determine the output to be supplied by each station by means of the load-duration-area curve, it becomes necessary to devise a common diagram in two dimensions, representing faithfully the daily and monthly variable load demand.

Fig. 13 shows a monthly load-duration-area curve or power-energy curve for the total demand of a power system. It is constructed by determining from the monthly load-duration curve the area, expressed in kilowatt-hours, above any load, P, and by plotting this value horizontally at the corresponding ordinate. The power-energy curve will be a straight line below the minimum load.

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[34] Asst. Hydr. Engr., The São Paulo Tramway, Light & Power Co., Ltd., São Paulo, Brazil.

The abscissas of the power-energy curve may be, not only in kilowatt-hours, but also in average kilowatts equal to the quantity of kilowatt-hours divided by the total time, in hours, for which the curve has been drawn. The abscissas, likewise, may be divided into average hours equal to the kilowatt-hours divided by the maximum yearly peak load, thus expressing the number of hours in which the maximum yearly peak would produce the same kilowatt-hours. The load factor, referred to the maximum peak, is obtained by dividing the average number of hours by the number of total hours of the power-energy curve. This curve, naturally, may be drawn for daily, weekly, monthly, or yearly loads, depending on the case at hand, which results in daily, weekly, monthly, or yearly average loads, hours, or load factors.

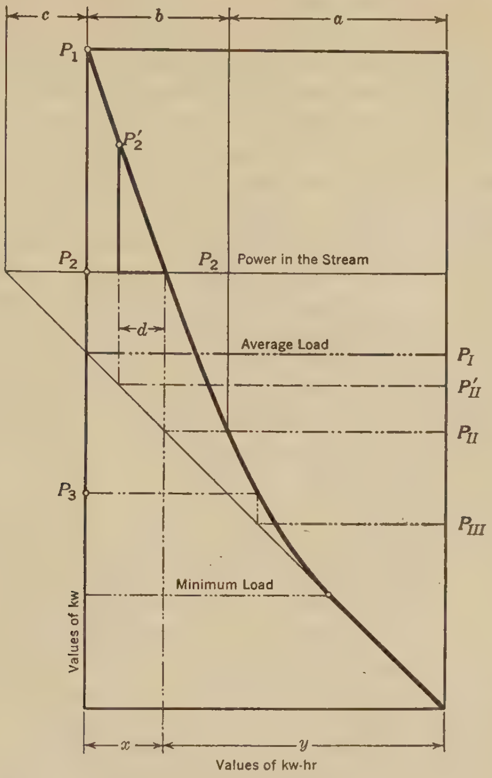

FIG. 13.—PROPERTIES OF DURATION-AREA CURVE.

Extending the straight part of the power-energy curve to the ordinate at the left, the intersection gives the average load, P_{I}. For any given load, P_2, the output of the system, equals y kw-hr, the deficiency, x kw-hr, and the waste, $(x + c)$ kw-hr, provided, the system load demand, y, below P_2 is supplied by a run-of-river plant without storage or pondage and with P_2 kw of power in the stream. With a storage of $(y - a)$ kw-hr, the same output, y kw-hr, is produced by P_{II} kw of power in the stream.

The yearly load "mountain" can now be reduced to a single diagram by using the properties of the power-energy curve. As the load, P_2, is equivalent to a monthly average load of P_{II} (for the case of Fig. 13), the same construction is repeated for other loads, P_1, P_2, P_3, etc., obtaining, finally, the reduced ordinates, P_I, P_{II}, P_{III}, etc., shown along the right side of Fig. 13. These reduced ordinates express the actual monthly load demand in terms of average load lasting one month. Hence, the load demand of one month is represented by a single vertical line with unequal sub-divisions which, in turn, are proportional to the monthly load factor. It should be noted, however, that the reduced ordinates, P_I, P_{II}, P_{III}, etc., have as parameters, the absolute values that correspond to the original ordinates, P_1, P_2, P_3, etc.

The required two-dimensional representation of the yearly system load-demand "mountain" is obtained by repeating this procedure for the other months (or weeks) of the year and arranging the reduced power-demand lines in chronological order, and then connecting the points with equal reduced ordinates. The area of this diagram, according to its construction, furnishes the yearly load demand of the power system. As long as the ordinates of the reduced power-demand lines with the parameters, P_1, P_2, P_3, etc., are as great as the corresponding ordinates along the left side of Fig. 13, the load is used 24 hours each day throughout the year.

Consider now a power system the base load of which is supplied by a run-of-river plant with, or without, pondage. The hydrograph of this plant is added to the load-demand diagram. For very flashy streams, it is convenient to strip the hydrograph of all flood flows that exceed a given value. This maximum flood to be represented can be determined easily according to the particular case; but as the power in the stream for the greatest part of the year is not used throughout the month it becomes necessary to reduce the hydrograph ordinates, expressed in kilowatts, to the monthly average load demand by projecting points with values of P_1, P_2, P_3, etc., on the hydrograph vertically downward to the corresponding load-demand lines, P_I, P_{II}, P_{III}, etc. This is the procedure, when no pondage is available at the run-of-river plant. With an available pondage of d kw-hr, P_2 kw of power in the stream are capable of supplying a maximum load of P'_2 kw, with an output of $(y + d)$ kw-hr. Reducing P'_2 kw to average load, a reduced ordinate (P'_{II}) is obtained which, naturally, is greater than the corresponding reduced load, P_{II}, for the case without pondage. With a maximum required pondage of x kw-hr, P_2 kw of power in the stream would produce the maximum required peak load, P_1, with the total output of $(y + x)$ kw-hr.

Next, connecting the points thus obtained on the load-demand lines, the reduced hydrograph as actually needed for power production (and, therefore, the yearly load-demand diagram with a correct comparison of supply and demand) is obtained. When constructed for one year, or for several years, this diagram permits reading (directly, or by means of mass curves) the energy produced by the run-of-river plant, the wasted and deficient energy, the energy to be supplied by one or several storage plants or steam plants, and that energy generated at the run-of-river plant with water from a storage plant above. The diagram shows also when the storage plants commence and cease operation. It shows also that deficiency and waste at the run-of-river plant may occur at the same time, provided no pondage is available. The diagram has some further advantages in that it shows clearly the relation between total system demand, and the energy generated by, and the capacity of, the run-of-river plant. The usual determination of the output of a run-of-river plant (without taking into consideration the relation between system demand and maximum plant capacity) gives output values that may be far too great. In design, the diagram assists greatly in constructing use curves, or determining utilization factors for various maximum plant capacities and system characteristics without the use of older and more tedious methods. The diagram may be completed and improved by introducing variable efficiencies of the plants and variations of head. The necessary modifications for this purpose are apparent and are omitted here for the sake of brevity.

The writer has found this diagram to be an exceedingly simple and fitting tool for all problems which when solved by the usual methods become tedious or are not sufficiently precise. Among other problems may be mentioned that of pumped storage working together with run-of-river plants with or without pondage, and monthly or yearly storage plants. Furthermore, the most economical distribution of peak and base load among the various power stations is capable of easy solution, which is especially valuable during dry periods and in those cases, where short-term weather forecasting is used.

A modification of the aforementioned method consists in using the duration-area curve of the supply instead of the hydrograph, which in certain cases is more advantageous.

In his previous paper[35] the author gave, for the modified Type I and Type III frequency curves, a very simple method for computing the ordinates of the duration curve, provided the coefficients of variation and skew are known. In the case of stream flow at least the duration curve gives a smooth continuous line, and the writer wonders why the results of the author's former paper have not been applied to the Butler, Tenn., record shown in Fig. 5 with many steps and irregularities. The writer is of the opinion that any stream-flow duration curve should be represented by a smooth line drawn either through the theoretical points, or through the points of individually plotted items or those plotted by groups, with the only restriction that the area below the curves thus obtained should be identical in the three cases. In most practical cases the theoretical duration curve for stream flow and Type III frequency curve are in sufficient agreement with the duration curve as obtained from the records.

The author's present paper deals with the application of duration curves to hydro-electric studies, but does not aim to discuss the mathematical theory of the duration curve by means of skew frequency curves. That was the purpose of the paper published in 1924, which reflects distinction on the author.

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[35] "Theoretical Frequency Curves and Their Application to Engineering Problems," by H. Alden Foster, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

In the present paper, Mr. Foster explains the duration curve by the roundabout method of the frequency curve. It would have been better to dismiss the frequency curve, especially as this curve, for the short-term records commonly available, is likely to be very irregular. For those engineers not familiar with the theory of skew frequency curves the simplest definition of the duration curve consists in stating that the sum of the abscissas of the histogram at any ordinate is shifted to the left, thus forming a continuous line, which expresses the number of days or years or the time in percentage during which the run-off or load was greater than a certain quantity. The frequency curve shows the number of days or years the item under consideration equalled a certain value. The frequency curve has only restricted application, in studies of the change in river bed, transportation of sedimentary material ("Geschiebe-förderung"), and inland navigation.

The four elements of hydro-electric studies (namely, the hydrograph, mass curve, duration curve, and duration-area curve, or its derivatives), each have their proper but limited application. The resourcefulness of the engineer will always find that combination of the four elements which gives a simple and comprehensive solution of a rather complicated problem, such as the demand and supply of an interconnected power system.

Although the paper deals only with the properties of the duration and duration-area curves and their application to simple hydro-electric studies, the author is to be commended for his timely paper as the duration-area curve, even in recent textbooks on hydro-electric engineering, has not been adequately discussed.

Dr. Dino Tonini[36] (by letter).—Much information concerning basic concepts that underlie duration curves is brought together in this paper. The application of this type of curve to hydrology is productive of interesting results and its detailed examination always discloses new and elegant characteristics.

As the author properly states it would be desirable that the time unit for original records be in terms of daily flow. Weekly and monthly duration curves can result in appreciable differences especially if the stream flow is regulated only by reservoirs of limited storage. Calculation with daily flow data is almost essential for plotting duration curves of short streams and, in general, for all water-sheds on which the flow is subject to sudden and considerable changes, such as "flashy" streams and those that are fed by melting snow and ice.

The duration curve, plotted by daily flow, in order of magnitude for a record of several years, may represent the "average year." Indeed, the duration curve for the entire record may be considered as the sum of different yearly curves and hence (apart from the time scale) as a duration curve of an "average year." The monthly duration curve, derived as an extension of the average duration curve, seems to have little value in practice. Essential variations in flows between one month and another are too important, having a physically definite character. For instance, it may be desired to know the average distribution of flow in the record of a certain month, say, January; then, the duration curve constructed from all available January records represents an "average January month." Of course, the sum of twelve similar average monthly curves gives an "average generic monthly curve" which is also that of an "average year"; but, as stated, it is inadequate for monthly computations since the distribution of run-off in different months is too variable. The duration curve, as an average curve, must be used carefully with records of less than a month; for instance, a duration curve showing the distribution of 24-hr average flows could not represent weekly distribution for glacial rivers, or for large water-sheds on which floods sometimes continue for many days, or for short rivers with sudden variations in flows, etc.

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If the storage reservoir is sufficient for 100% regulation, the regulated-flow duration curve is a horizontal line equal to the average discharge of record. On the other hand, the practical development of storage is generally less than that required for complete regulation; hence, the less storage there is available, the more the regulated-flow curve will depart from the straight line of average flow. More important than to provide for a minimum regulated flow, is the problem of securing the best regulation with a given storage reservoir, thus using as much of the natural stream flow above the minimum as is possible with a power plant of limited size. In other words, it is necessary to find, for every available storage, the regulation that permits the maximum utilizaton of natural run-off, with the minimum power plant.

Duration curves and duration-area curves have been used by the Hydrographic Service of Italy for a long time. This Service has also computed all the hydraulic power resources of the Kingdom of Italy, basing the estimates in particular on the application of duration, duration-area, and mass curves. As a result, the officers of the Service, and private engineers as well, have described various mathematical and graphical properties of these curves.

The first application of the duration-area curve seems to have been made in 1920 by a French engineer, A. Coutagne.[37] The Italians, however, took a great interest in the subject, and developed it independently.[38] In Italy, the duration-area curve is called the "hydrologic characteristic of power development."

If q is the run-off and t, its respective duration, the curve, $q = f(t)$, is the duration curve of run-off (generally constructed for data in decreasing order of magnitude (see Fig. 14)). The curve,

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[37] "Considérations sur les éléments caractéristiques du règime d'un cours d'eau," by A. Coutagne, *Revue Générale d'Electricité,* August 28, 1920.

[38] "I cicli delle portate naturali dell 'Adda Alpina," by Prof. G. Fantoli, *Annali dei Lavori Pubblici,* November, 1926, and September, 1927; "Preliminare esame comparativo delle condizioni idrologiche delle varie regioni italiane," by Prof. G. De Marchi, Relazione del Servizio Idrografico Centrale, Vol. I and III, Memorie e Studi idrografici del S. I.; "Sulla capacità da assegnare ai serbatoi stagionali," by Prof. P. Frosini, *Annali dei Lavori Pubblici,* September, 1928; also, by the same author, "Determinazione della capacità da assegnare ai serbatoi per uso irriguo," *Annali dei Lavori Pubblici,* August 1929; "Su un procedimento statistico applicabile a determinazioni idrologiche," by Prof. L. Gherardelli, *Annali dei Lavori Pubblici,* May, 1929; also by the same author, "Criteri per la regolazione parziale dei bacini imbriferi," *Annali dei Lavori Pubblici,* July, 1929; "Applicazione di metodi statistici all'idrologia," by Prof. Di Ricco, *Bollettino N. 8* del Comitato Nazionale Italiano Geodetico e Geofisico, Venezia, 1924.

$$y = \int_0^q t \, dq \quad \text{.........................} (7)$$

is the duration-area curve, plotting values of q on the axis of abscissas and values of the integral Equation (7), on the axis of ordinates (see Fig. 14(*b*)). The function,

$$y = \int_0^t q \, dt \quad \text{.........................} (8)$$

on the other hand, is a special well-known curve termed a "concentration curve," from which the "ratio of concentration" may be obtained. Theoretically, the latter is equal to zero for a uniform distribution of flow in the period of record, and approaches unity as its upper limit as more of the recorded flow is concentrated in a small part of the entire period.

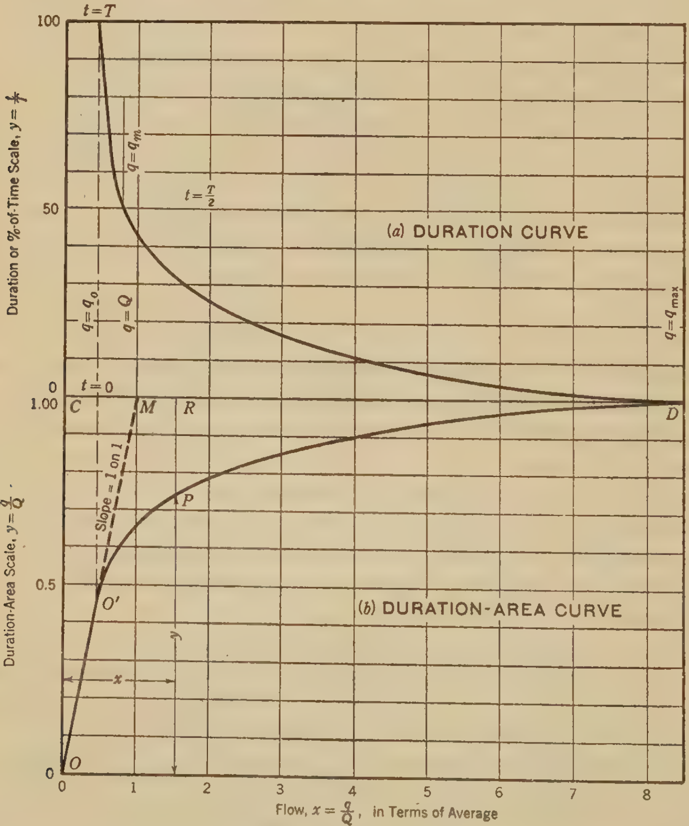

FIG. 14.

Professor Gherardelli[38] has described some new properties of the duration and duration-area curves. He uses the "%-of-time" method for plotting the duration curve, and gives flows in terms of their mean. The abscissas, x, and the ordinates, y, of the duration-area curve may then be defined by the following equations:

$$x = \frac{q}{Q} \quad \text{.........................} (9a)$$

$$y = \frac{1}{TQ}\int_0^q t\,dq = \frac{\overline{q}}{Q} \quad \text{.....................} (9b)$$

in which,

$$\overline{q} = \frac{1}{T}\int_0^q t\,dq \quad \text{.........................} (9c)$$

q is the rate of flow; Q, the average or mean flow; t, the duration of the flow; and T, the period of record.

In Fig. 14 (*b*), the duration-area curve which intersects the x-axis at a slope of 1:1, is tangent to the straight line, *OM*, at *O′*, where $x = \frac{q_0}{Q}$ (q_0 = minimum flow). The curve is also tangent to the horizontal line, *CD* ($y = 1.0$), parallel to the x-axis, at a point, *D*, where $x = \frac{q_{\max}}{Q}$ ($q_{\max}$ = maximum flow). The derivative, $\frac{dy}{dx}$, at any point, x, gives the duration (%-of-time) of the flow, q ($= xQ$); the second derivative gives its frequency.

At a given point, *P*, of the duration-area curve, the ordinate, y, determines the fraction of average stream flow available with a canal of the maximum capacity, xQ. The segment, *PR* ($= 1 - y$), represents lost run-off; while the various segments parallel to the axis of ordinates and situated between the curve, *OD*, and the line, *OMD*, give the surplus of run-off available with storage reservoirs.

The area, *O′MD*, has an interesting statistical meaning, because it is a function of the relative standard deviation of flow; in fact,

$$\text{Area } O'MD = \text{Area } OCD - \text{Area } OCM$$

From Equations (9),

$$\text{Area } OCD = \int_0^1 x\,dy = \frac{1}{TQ^2}\int_0^{q\,(\max)} q\,t\,dq$$

and from the duration curve, $q = f\,(t)$ and $dq = f'\,(t)\,dt$. Hence,

$$\text{Area } OCD = \frac{1}{TQ^2}\int_T^0 q\,f'\,(t)\,t\,dt = \frac{1}{TQ^2}\int_T^0 \frac{d}{dt}\left(\frac{q^2}{2}\right)t\,dt$$

or,

$$\text{Area } OCD = \frac{1}{TQ^2}\left(\left[\frac{q^2}{2}\,t\right]_T^0 + \int_0^T \frac{q^2}{2}\,dt\right)$$

The expression, $\left[\frac{q^2}{2}\,t\right]_T^0$, is equal to zero, because when $t = T$, $q = 0$; and when $t = 0$, $q = q_{\max}$. Therefore,

$$\text{Area } OCD = \frac{1}{TQ^2}\int_0^T \frac{q^2}{2}\,dt$$

Now, since Area $OCM = \frac{1}{2}$, it follows that,

$$\text{Area } O'MD = \frac{1}{TQ^2}\int_0^T \frac{q^2}{2}\,dt - \frac{1}{2} \quad\ldots\ldots\ldots\ldots\ldots(10)$$

The relative standard deviation from the arithmetical average of flows can be computed by the duration curve. The sum of the squares of the deviations is:

$$\int_0^T (q - Q)^2\,dt = \int_0^T q^2\,dt - 2\,Q\int_0^T q\,dt + Q^2T$$

Since $\int_0^T q\,dt = Q\,T$,

$$\int_0^T (q - Q)^2\,dt = \int_0^T q^2\,dt - Q^2T$$

Then, letting s equal the relative standard deviation,

$$s^2 = \left(\frac{\text{Standard deviation}}{Q}\right)^2 = \frac{\int_0^T (q - Q)^2\,dt}{Q^2T}$$

or,

$$s^2 = \frac{1}{TQ^2}\int_0^T q^2\,dt - 1 = 2\text{ Area } O'MD \quad\ldots\ldots\ldots\ldots(11)$$

From Equations (10) and (11), it follows that Area $O'MD$ is equal to one-half the square of the relative standard deviation from the arithmetical average.

For the duration-area curve Professor Fantoli[38] gives the equation,

$$y = 1 - e^{-r}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(12)$$

in which, $r = \frac{x}{n}$, x and y are the same as before, and n is a numerical coefficient. (It varies from 0.77 for the Tevere River to 1.80 for the Arno River.) This coefficient is also represented by Area OCD; for,

$$\text{Area } OCD = \int_0^\infty (1 - y)\,dx = \int_0^\infty e^{-r}\,dx = n \quad\ldots\ldots\ldots(13)$$

The coefficient is also a function of the relative standard deviation, s. From Equations (10), (11), and (13), it follows that $s = \sqrt{2n - 1}$. Fantoli's formula, Equation (12), is in fairly close agreement with the actual curves.

Coutagne has also considered these matters. For the duration curve he proposed the following:

$$\frac{q - q_0}{Q - q_0} = (N + 1)\left(\frac{T + t}{T}\right)^N \quad\ldots\ldots\ldots\ldots\ldots\ldots(14)$$

The factor, N, is a new "irregularity coefficient," determined by the flow,

$q = q_m$ when $t = \frac{T}{2}$; thus,

$$\frac{q_m - q_o}{Q - q_o} = \frac{N + 1}{2^N} \qquad (15)$$

Theoretically, for a perfectly regular flow, $N = 1$; but in practice it varies between 2 and 5.

The equation for the duration-area curve proposed by Coutagne (using $x = \frac{q}{Q}, y = \frac{\overline{q}}{Q}$) is:

$$y = x - \frac{N\,(x - q_o)^{1+z}}{(N + 1)^{1+z}\,(Q - q_o)^z} \qquad (16)$$

in which (to simplify typography), $z = \frac{1}{N}$.

The theoretical curve, Equation (12) or Equation (16), must be tangent to the line, $x = y$, at the point, $x = y = q_o$; and, also, tangent to the line, $y = 1$ $(\overline{q} = Q)$, at the point, $x = x_{\max}$ $(q = q_{\max})$. Hence, an approximate duration-area curve can be constructed quickly as a parabola of the second degree fully defined by these points and their tangents (method of tangents).

Another type of empirical duration and duration-area curves is developed from probability curves, as proposed by a French writer, R. Gibrat.[39] Further interesting applications of the duration and duration-area curve are given by a German author, A. Ludin.[40]

H. Alden Foster,[41] M. Am. Soc. C. E. (by letter).—In the introduction to the paper, the writer indicated that his object was to present a general description of the fundamental properties of duration curves, with the hope that the discussions would bring out additional practical applications. The results have thoroughly justified that hope. The paper has brought out much new information concerning duration curves which, so far as the writer is aware, has not heretofore been published in American engineering literature.

Several of the discussers express regret that the writer apparently had abandoned the use of theoretical frequency and duration curves, and thus appeared somewhat in the light of an apostate. This, however, is far from the truth. The writer is as fully convinced now, as he was in 1924, of the usefulness of theoretical frequency curves for engineering studies. There is no doubt that such methods can be applied advantageously to many types of duration diagrams; but the development of these curves has been rather fully covered by other papers, as has already been noted. In preparing the present paper, the writer purposely avoided any digression from the study of duration curves as actually constructed from observed data. If the fundamental properties of such curves are well understood, the use of theoretical curves should be that much easier.

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[39] "Aménagement hydroélectrique des cours d'eau; Statistique mathématique et calcul des probabilités," by R. Gibrat, *Revue Générale de l'Electricité*, October 15, 1932.

[40] "Bedarf und Dargebot," by A. Ludin, J. Springer, Berlin, 1932.

[41] Res. Engr., Parsons, Klapp, Brinckerhoff & Douglas, North Platte, Nebr.

Two uses of duration curves were indicated in the paper. Colonel Pettis calls these "flood-duration curves" and "power-duration curves." The writer would prefer not to use any such distinctive names, as all the curves are fundamentally of the same type. The difference is only in the manner in which they are used. The theoretical curves are particularly useful for probability studies, but if the data can be presented suitably by a smooth curve, there is no reason why the latter should not be used as a picture of what the data would be expected to give if they were more extensive.

The important point is, that a duration curve plotted from the original records, as explained in the paper, can be used to obtain results in actual agreement with those records. If it is desired to obtain results to correspond with what should be expected from a record of infinite length, then the theoretical curve should give more reliable results.

Comparison of Daily and Monthly Flow-Duration Curves.—Mr. Pfaehler, Mr. Hoyt, and Mr. Watson have commented on the error involved by use of a curve based on monthly average flows instead of daily averages. The writer has given considerable attention to this question, which seems worthy of further discussion. The error will always be such as to give results which are too large as compared with what would be obtained from the daily flow-duration curve. An examination of Fig. 6 shows that the area beneath the monthly curve, up to any assumed rate of discharge, is greater than the corresponding area under the daily curve, and this inequality will hold for all rates of flow up to the maximum daily discharge.

A study has been made by the writer to determine the relative amount of this error for a particular stream. Flow-duration curves obtained from the records at three gauging stations on the Delaware River water-shed were used. (The daily flow-duration curves were computed by Mr. E. Laurence Burnett, together with the duration-area values for various rates of stream flow.) The ratio of the daily to the monthly duration-area value was then derived. Fig. 15 with Table 2 shows the variation of this ratio plotted against the stream flow expressed in cubic feet per second per square mile. The curves show that the ratio of the daily to monthly duration-area (or average run-off) value begins at unity for a certain low rate of discharge, which is the minimum daily flow on record. The ratio decreases for higher rates of discharge, and reaches a minimum at about 1½ times the mean stream flow. Beyond that, the ratio increases until it finally reaches unity again at a discharge equal to the maximum daily flow on record.

The three curves in Fig. 15 are for records at points of greatly different drainage area and average stream flow, but they are very similar in shape. It is evident that as the drainage area increases, the ratio values also increase; but the curves for Port Jervis, N. Y., and Cook's Falls, N. Y., are almost identical for flows up to mean run-off, and even the curve for Trenton, N. J., is only 2% or 3% higher than the other two, for rates of discharge less than the mean. Apparently, the curve for Port Jervis could be used without serious error for practically all points on the water-shed farther up stream (except at very high rates of discharge), and also for a considerable portion of the water-shed below Port Jervis.

It is suggested, therefore, that when any extensive study is to be made on a given water-shed, at least one flow-duration curve be computed from a daily record. Satisfactory results should be obtained with a record as short as ten years. The corresponding monthly duration curve should also be computed, for the same period of record. From the resulting duration-area values, the ratios of daily to monthly average run-off are computed, and plotted against the stream flow, in cubic feet per second per square mile. Then, when the record at some other station on the water-shed is to be studied, only the monthly flow-duration curve, and duration-area curve, need be computed The latter can be reduced to an equivalent daily duration-area curve by multiplying by ratios obtained from the curve already plotted. The results should be reasonably precise, even on a large water-shed; and if two daily curves are obtained at points of large and small drainage area, the corresponding ratio curves can be interpolated for intermediate points. The final results should be accurate within a few per cent.

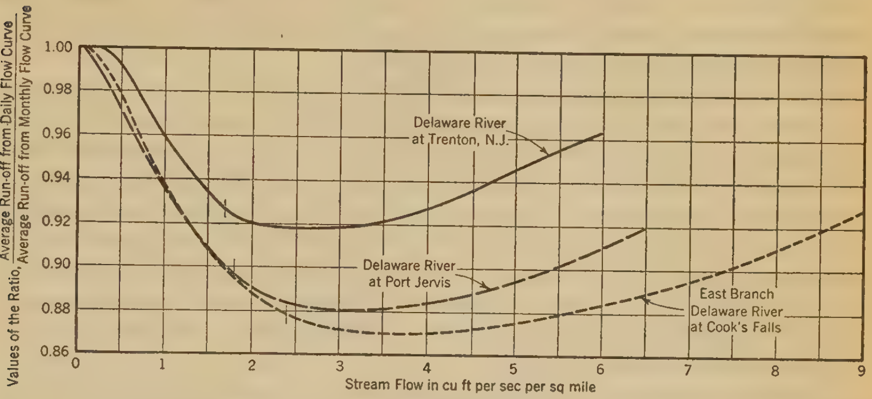

FIG. 15.—COMPARISON OF DAILY AND MONTHLY FLOW-DURATION CURVES, DELAWARE RIVER WATER-SHED

TABLE 2.—COMPARISON OF DAILY AND MONTHLY FLOW-DURATION RECORDS, DELAWARE RIVER WATER-SHED

Location	Period of record	Drainage area, in square miles	Average stream flow from the record, in cubic feet per second	Stream flow, in cubic feet per second per square mile
Trenton, N. J.	1913–1930	6 800	11 500	1.69
Port Jervis, N. Y.	1904–1930	3 070	5 600	1.82
Cook's Falls, N. Y.	1904–1930	241	580	2.40

The shape of the ratio curves will doubtless vary for different water-sheds, as the minimum value of the ratio should be smaller for streams which have rapid variation in daily flow (that is, "flashy" streams). The fact that the larger drainage area results in higher ratios is to be expected, since an increase in drainage area has the effect of "dampening" the effect of sudden storms which would cause large daily fluctuations on a small drainage area.

The reason for plotting the stream flow in cubic feet per second per square mile, instead of in terms of mean run-off, is to reduce the effect of variation in drainage area. Since the average flow in cubic feet per second per square mile decreases as the drainage area increases, the ratio curves are brought closer together than they would be if the mean run-off method of plotting were used. However, if only one ratio curve is to be plotted for a given water-shed, either method of plotting would be satisfactory.

The writer wishes to emphasize the importance of making some correction to the monthly flow-duration curve. The present study shows that the resulting error in average flow below a given discharge may be 15%, or more. If it is impossible to construct a daily flow-duration curve for the stream under investigation (due to lack of records, or limited time), a ratio curve derived from the daily records on some other stream could be used as a guide in obtaining a set of correction factors, which would then be applied to the monthly duration curve on the stream in question. The results, even if not very precise, would at least be more conservative than would be obtained from the monthly curves without correction.

Having obtained an equivalent duration-area curve of daily flow, as described previously, it is a simple matter to derive the corresponding flow-duration curve, by means of the slope of the duration-area curve, merely reversing the process explained for the "Duration-Area Curve."

Historical Development.—Several of the discussions present additional information regarding the history of duration curves. Mr. Pfaehler states that he used flow-duration curves in 1908 and 1909. Mr. Sargent gives further reference to his own work on the subject, and shows that a form of duration-area curve was used by him in 1911. Dr. Tonini states that this latter curve was introduced in France in 1920 by Coutagne, and that it has been extensively used in Italy in recent years. Mr. Cook's reference to the paper[30] by the late Mr. Herschel, published in 1878, is of particular interest.

Comments on Discussions.—Mr. Pfaehler has shown how a one-year duration curve may be of value to indicate special conditions in years of drought or excessive flow. He considers the duration curve of regulated flow as not being suitable for use in complicated stream-flow and reservoir studies. This is a question that must be decided by the engineer in any particular case. In fact, as has been mentioned also by Mr. Knapp, each of the graphical methods of studying stream-flow records has its own advantages and limitations. It is only through detailed knowledge of the various methods that an engineer is enabled to select those that are best adapted for use in analyzing a given problem.

Mr. Sargent's statement that computations of power output from duration curves should be discounted is of interest in connection with Mr. Knapp's discussion of this matter.

Colonel Pettis' discussion is chiefly an outline of his methods for determining the probability of floods. It should be of value in bringing his interesting work to the attention of the profession, since this is the first time that his theory has been described in any publication of the Society, so far as the writer is aware. Since the question of floods is very broad in itself, and is somewhat outside the scope of the present paper, the writer will not attempt any further comment.

Mr. Hoyt's discussion is particularly valuable in its explanation of the use of duration curves for international comparisons of river flow and available power. He makes a distinction between "duration curves" and "deficiency curves." The writer would prefer to use the term, "duration curve," in a more general sense, so that the deficiency curve would be a special type of duration curve. The writer has always thought of the duration curve as related to its statistical origin. From this point of view, any set of data which can be arranged in a frequency table can be represented by a duration curve; and a monthly or yearly average-flow record can be presented as a duration curve just as logically as a daily or hourly flow record. The essential point in showing such duration curves is, as stated by Mr. Hoyt, that "the explanation must always indicate the time unit used."

Mr. Hoyt states that the duration curves of daily and monthly flow can be used to correct the storage computed from a mass curve based on monthly data. Storage computations by means of the mass curve are generally based on monthly average flows, as the plotting of a mass curve for daily flows is very laborious. The results, however, always involve a certain error, which depends on the distribution of daily flows during the last month of the period of maximum reservoir drawdown. The writer has indicated in the paper that the duration curve by itself does not show the amount of storage actually required. Hence, the difference between the daily and monthly duration curves will not show, theoretically, the proper correction to be applied to storage as computed from the monthly mass curve. However, it is quite probable that the error involved in this method is small, especially since the proper correction to the mass-curve storage value is itself probably within the limits of 10 to 20 per cent. Accordingly, the engineer should be justified in determining this correction for "daily storage"[42] by means of the daily and monthly duration curves, which should give a closely approximate value and should be considerably easier than using a daily mass curve. The writer is indebted to Mr. Hoyt for suggesting this method.

Mr. Santos has given a clear explanation as to how a theoretical duration curve may be computed from the observed data, on which the writer has already commented herein. While a discussion of theoretical curves is outside the limits which the writer intended to set for the paper, reference will be made to Mr. Santos' suggestion whether some other form of curve than Pearson's Type I or Type III might afford a better fit in certain cases. This is doubtless true. It was shown in the paper to which Mr. Santos refers[21] that the Type I and Type III curves of Pearson could be readily computed by means of tabulated factors, using only values for the coefficient of variation and coefficient of skew determined from the recorded data. The method given required no knowledge of the mathematics of skew probability curves.

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[42] See "Storage to Be Provided in Impounding Reservoirs," by the late Allen Hazen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1573.

Pearson gave seven types of curves, of which Type II is a special case for Type I, and Type VII is a special case for Type III. Types IV, V, and VI are of a form not commonly met in engineering work, and, moreover, are difficult to reduce to tabular form as was done for Types I and III. It is quite possible that some classes of engineering data might be represented more accurately by these other types; but the computation of the curves is probably outside the ability or patience of most engineers, when tabulated factors are not available.

Mr. Knapp has given an excellent description of the application of the duration-area, or "power-energy," curve to problems of load variation and power output, which requires no additional comment. Of particular significance is his explanation of the use of this curve in determining the output of a hydro-electric plant as affected by the demand requirements. As Mr. Sargent has noted, the duration curve by itself is likely to give results in power output greater than can actually be used. Mr. Knapp would use the theoretical duration curves in preference to observed data, but would treat the duration curve without reference to its related frequency curve. It is interesting to compare his comments in this respect with Mr. Cook's discussion of the properties of frequency curves.

Mr. Watson gives an interesting comparison of the results of computing curves by the "total-period method" and the "calendar-year method." The writer would prefer the total-period method as being more accurate, although it is possible that the second method might be satisfactory in some cases, especially when adjusted by the methods described in the paper to which Mr. Watson refers.[34] His comments on the relative accuracy of daily and monthly flow-duration curves are in close agreement with the writer's conclusions.

Mr. Cook has given a valuable description of the frequency curve, its relation to the duration curve, and its interpretation as regards physical conditions. He notes the difficulty in locating the mode correctly. In this connection, if the Pearson Type I or Type III curve is used, the mode can be located readily by the following formulas:[43] If d = Mean — Mode,

For Type I,

$$d = (cs) \times (cv) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

For Type III,

$$d = \frac{(cs) \times (cv)}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Equations (17) and (18) give the distance by which the mode comes below the mean (on the stream-flow axis), expressed in terms of mean stream flow. For the Type III curve, the difference between the mean and the median[44] is approximately equal to $\frac{d}{3}$

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[43] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 153, 186.

[44] *Loc. cit.*, p. 157.

Mr. Cook's remarks on the danger of using too short a time unit in certain cases are significant, and should be compared with Mr. Jarvis' statements regarding the Upper Mississippi River.

Mr. Mattern does not consider the duration curve suitable for studies of power plant operation, in which respect he seems to be in agreement with Mr. Pfaehler. The writer has already commented on this question. Mr. Mattern's suggestion that, because the hydrograph is easier to explain to the layman, it should be used in preference to the duration curve, does not seem fully justified. By the same argument, the engineer should not use calculus in his computations because the president of the company does not understand an integral sign. If the duration curve is helpful to the analysis, there is no reason why it should not be used; if not, then some other method should be adopted in preference.

Dr. Tonini has rendered a valuable service to American engineers by bringing to their attention the work of his compatriots and of other European engineers on the development of duration and duration-area curves. His analysis of the properties of the duration-area curve is particularly valuable. In this connection, it may be noted that the "relative standard deviation from the arithmetical average of flows" is the same as the "coefficient of variation" as used in American practice.

The empirical equations for duration and duration-area curves, quoted by Dr. Tonini, are interesting and may prove of value in some cases. The writer, however, would prefer to avoid the use of such formulas, as he believes greater accuracy will be obtained in most cases by duration curves of the Pearson types, in which the constants are obtained from the observed data in each particular case. The references noted by Dr. Tonini should be useful, particularly the pamphlet by A. Ludin,[40] which contains many interesting suggestions for the graphical analysis of stream-flow data by the hydrograph, mass curve, and duration and duration-area curves.

In conclusion, the writer wishes to express appreciation of the interest shown in this paper, and for the many valuable suggestions and methods which have been made available to the profession by the several discussions.

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Paper No. 1886

ANALYSIS OF UNSYMMETRICAL CONCRETE ARCHES

BY CHARLES S. WHITNEY[1], M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. DAVID A. MOLITOR, AND CHARLES S. WHITNEY.

SYNOPSIS

In the design of multiple-span arch bridges it is frequently found desirable to raise or lower the abutment end of the end spans to produce the most favorable abutment conditions. The end spans then become unsymmetrical arches which require special analysis.

This paper presents an extension to unsymmetrical arches of the method in the writer's paper[2] entitled, "Design of Symmetrical Concrete Arches," which was based on the work of Strassner.[3] It permits the construction of influence lines for the reactions of an unsymmetrical arch. An attempt is made to clarify the action of the unsymmetrical arch so that each step can be readily understood by the designer.

The simplicity of this method of analysis, which is also quite general in its application, permits a more thorough study of the proportions of an unsymmetrical arch which should result in a more satisfactory design. This paper is concerned only with what has been termed the "geometry" of the problem and does not include the selection of values of modulus of elasticity, movement of abutments, coefficients of thermal expansion, plasticity, shrinkage, etc., although formulas are given to determine their effects on the reactions. The full investigation upon which this paper was based, has been placed on file in Engineering Societies Library, in New York, N. Y.

NOTE.—Published in October, 1933, *Proceedings*.

[1] Cons. Engr., Milwaukee, Wis.

[2] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 931.

[3] "Neuere Methoden zur Statik der Rahmentragwerke," Vol. II, by A. Strassner, Berlin, Wilhelm Ernst & Sohn.

INTRODUCTION

For the purpose of analysis the unsymmetrical arch is divided at the crown point into two parts and the elastic properties of each part are determined separately, after which the action of the entire rib is considered. By using the formulas for arch-axis co-ordinates and rib-thickness variation given in the previous paper,[2] tables have been prepared which give these elastic properties without any involved calculations. In order to avoid lengthy repetition, reference is made to that paper for derivation of basic formulas and a full discussion of their application.

It should be noted that although the tables and diagrams given herein to assist in design are based on certain assumptions regarding the arch-axis curve and the rib-thickness variation, the method of analysis is general and the formulas for reactions can be applied to any arch rib by performing the integrations either graphically or numerically using the actual rib dimensions.

The treatment is arranged in three parts, giving first, the elastic properties of the rib segments to each side of the crown; second, the reactions of the complete rib; and third, a numerical example and a brief discussion of unsymmetrical arch design.

I.—ELASTIC PROPERTIES OF RIB SEGMENTS

Equation of Arch Axis.—In order to permit an accurate mathematical analysis formulas for the co-ordinates of the arch axis and the variation of moment of inertia along the rib have been derived by the writer in his previous paper entitled "Design of Symmetrical Concrete Arches".[2] The arch-axis formula is a transformed catenary giving a curve that lies very close to actual dead load pressure lines. Its form is dependent on the ratio of w_s, the dead load per linear foot at the springing, to w_c, the dead load at the crown. For the special case of $w_s = w_c$, the axis becomes parabolic.

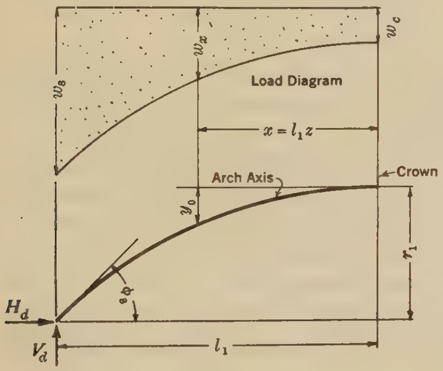

FIG. 1.

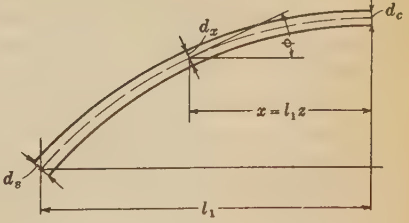

FIG. 2.

It is necessary to consider concentrated loads as distributed in determining the axis curve, but the effects of the concentration can be determined from the influence lines.[4]

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[4] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), pp. 970, 1089–1092, inclusive.

The general formula[5] for the arch axis is (see Fig. 1),

$$y_0 = \frac{r}{g-1} (\cosh z\,k - 1) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, $g = \dfrac{w_s}{w_c} = \cosh k$; or, $k = \cosh^{-1} g$.

The ordinate, when $x = \dfrac{l_1}{2}$ (the quarter-point ordinate), is a direct function of g and can be used instead of g to establish the curve if desired. The ratio of this ordinate to the rise, r, is designated as N. When $g = 1$, the curve is a parabola and $N = 0.25$.

The dead load reactions can be computed by the formulas:

$$V_d = w_c\, l_1 \frac{\sqrt{g^2 - 1}}{k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

and,

$$H_d = w_c\, l_1^2 \frac{(g-1)}{r_1\, k^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, the subscript, 1, indicates the left-hand segment of the rib and the subscript, 2, indicates the right-hand segment. It is assumed in all formulas that the proper values of g, k, l_1, l_2, etc., will be used, depending on which segment is being considered. Tables giving the values of $\dfrac{y_0}{r}$, N, g, H_d, V_d, and tan ϕ_s, will be found in the previous paper.[6]

With the unsymmetrical arch, different curves will be used for the two sides because the dead load at one end will not be the same as that on the other.

If the roadway is nearly level as in most cases, the arch-axis formula can be applied to the two sides independently after the crown point has been located so that the horizontal thrusts are balanced.

If the slope of the roadway is unusually steep it may be best to determine the dead load pressure line graphically from the actual loading and to measure the "quarter-point" ordinates to determine the proper values of N. The curves of the axis on each side can then be established from the respective values of N. The arch-axis curve will then pass through the dead load pressure curve at five points, crown, quarter-points, and springings, and will ordinarily correspond satisfactorily for the full span.

After the rise and loading of the two segments of the arch have been determined, the horizontal position of the crown point can be located by using Equation (3) and placing H_d for the left segment equal to H_d for the right segment.

Variation of Thickness of Rib.—In order to determine the reactions of an arch, it is necessary to assume the relative values of the moments of inertia of the cross-section of the rib at the crown and at the springings. Since the two segments are considered separately, it is not necessary to have the section at the left springing the same as that at the right. The variation in moments of inertia in both segments from crown to springing is assumed to follow the relation,[7]

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[5] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), Equation (38), p. 968.

[6] *Loc. cit.*, Table 6, p. 973; and Table 9, p. 976.

$$I_x = \frac{I_c}{[1 - (1 - m)\, z] \cos \phi} \qquad (4)$$

in which, I_c = the moment of inertia of crown section; I_x = the moment of inertia of section at distance, x; I_s = the moment of inertia at springing; and, $m = \frac{I_c}{I_s \cos \phi_s}$, or $\frac{d^3_c}{d^3_s \cos \phi_s}$ for plain rectangular sections.

If it is assumed that the moment of inertia of the rib varies as the cube of the thickness, Equation (4) can be changed (see Fig. 2) to:

$$d_x = d_c\, c \sqrt[6]{1 + \tan^2 \phi} \qquad (5)$$

in which,

$$c = \frac{1}{\sqrt[3]{1 - (1 - m)\, z}} \qquad (5a)$$

The rib thickness at any point can then be obtained from Equation (5) with the help of Tables 12 and 13 of the writer's previous paper.[8] The value of $\cos \phi_s$ can also be computed from Table 12 of that paper, with the formula,

$$\cos \phi_s = \frac{1}{\sqrt{1 + \tan^2 \phi_s}}$$

When the reinforcement of the rib is such that I does not vary as the cube of d, the value of m in Equation (5) should be computed from d_c and d_s, but in formulas giving elastic properties or reactions the actual values of I_c and I_s should be used.

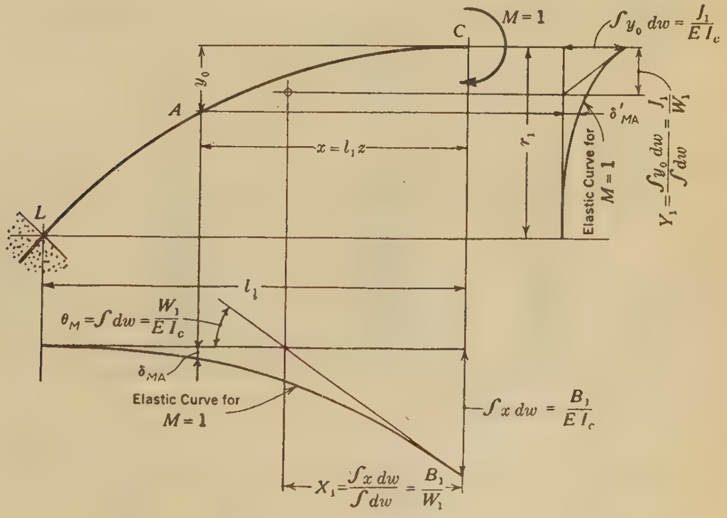
C
M = 1
$\int y_0\, dw = \frac{J_1}{E I_c}$
y_0
A
$x = l_1 z$
δ'_{MA}
r_1
Elastic Curve for M = 1
$Y_1 = \frac{\int y_0\, dw}{\int dw} = \frac{J_1}{W_1}$
L
l_1
$\theta_M = \int dw = \frac{W_1}{E I_c}$
δ_{MA}
Elastic Curve for M = 1
$\int x\, dw = \frac{B_1}{E I_c}$
$X_1 = \frac{\int x\, dw}{\int dw} = \frac{B_1}{W_1}$

FIG. 3.

Elastic Properties of Rib Segments.—Consider the arch cut at the crown with the right half removed, leaving the left segment projecting as a canti-

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[7] *Transaction*, Am. Soc. C. E., Vol. 88 (1925), p. 978.

[8] *Loc. cit.*, pp. 980–981.

lever from the abutment, as shown in Fig. 3. Formulas will be derived for the elastic deformations of the rib segment produced by unit horizontal and vertical loads and a unit moment applied at the crown point, C, which is the free end of the cantilever.

Using equations for the arch axis and the rib thickness, previously given, the value of $\frac{ds}{E\,I}$ (designated by dw), can readily be derived.[9] The formula is, as follows:

$$dw = \frac{l_1}{E\,I_c}\left[1 - (1 - m)\,z\right] dz \;\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The angular displacement at C produced by a unit moment is:

$$\int M\,dw = \int dw = \frac{l_1}{E\,I_c}\int\left[1 - (1 - m)\,z\right].dz = \frac{l_1}{E\,I_c}\,\frac{1 + m}{2} \;\ldots.(7)$$

or,

$$\int dw = \frac{W_1}{E\,I_c} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7a)$$

in which, W_1 is a constant for each particular rib segment which can be computed from the values given in Table 1. The vertical displacement at C produced by the unit moment, is:

TABLE 1.—VALUES OF $\int dw$, $\int x\,dw$, $\int x^2\,dw$, AND X_1

(FOR ONE SEGMENT OF ARCH RIB).

(All intermediate values by interpolation)

Value of m	$\frac{W_1}{l_1}$ Equation (7)	$\frac{B_1}{l^2_1}$ Equation (8)	$\frac{D_1}{l^3_1}$ Equation (13)	$\frac{X_1}{l_1}$ Equation (10)
0.15	0.575	0.2167	0.1208	0.3768
0.20	0.600	0.2333	0.1333	0.3889
0.25	0.625	0.2500	0.1458	0.4000
0.30	0.650	0.2667	0.1583	0.4103
0.35	0.675	0.2833	0.1708	0.4198
0.40	0.700	0.3000	0.1833	0.4286
0.45	0.725	0.3167	0.1958	0.4368
0.50	0.750	0.3333	0.2083	0.4444

$$\int M\,x\,dw = \int x\,dw = \frac{l^2_1}{E\,I_c}\int\left[z - z^2 + m\,z^2\right] dz = \frac{l^2_1}{E\,I_c}\,\frac{1 + 2\,m}{6} \;\ldots(8)$$

or,

$$\int x\,dw = \frac{B_1}{E\,I_c} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8a)$$

in which, $B_1 = E\,I_c \int x\,dw$, and can be computed from values in Table 1.

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[9] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 983.

The horizontal displacement at C produced by the unit moment is,[10]

$$\int M\,y_0\,dw = \int y_0\,dw = \frac{l_1\,r_1}{E\,I_c\,(g-1)}$$

$$\left[\frac{\sqrt{g^2-1}}{k} - 1 - (1-m)\left(\frac{\sqrt{g^2-1}}{k} - \frac{g-1}{k^2} - \frac{1}{2}\right)\right] \ldots\ldots (9)$$

or,

$$\int y_0\,dw = \frac{J_1}{E I_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9a)$$

in which, $J_1 = E\,I_c \int y_0\,dw$, and can be obtained from Table 2(*a*).

TABLE 2.—FACTORS FOR THE SOLUTION OF EQUATIONS (9), (11), (14), AND (16).
(Intermediate values by interpolation)

N	$m = 0.15$	$m = 0.20$	$m = 0.25$	$m = 0.30$	$m = 0.35$	$m = 0.40$	$m = 0.45$	$m = 0.50$
(*a*) VALUES OF $\frac{J_1}{l_1 r_1}$ (EQUATION (9))								
0.25	0.1208	0.1333	0.1458	0.1583	0.1708	0.1833	0.1958	0.2083
0.23	0.1142	0.1262	0.1383	0.1503	0.1624	0.1744	0.1865	0.1985
0.21	0.1074	0.1190	0.1305	0.1421	0.1537	0.1653	0.1769	0.1885
0.19	0.1006	0.1117	0.1228	0.1339	0.1450	0.1561	0.1673	0.1784
0.17	0.0936	0.1042	0.1149	0.1255	0.1361	0.1468	0.1574	0.1680
0.15	0.0866	0.0968	0.1068	0.1170	0.1271	0.1372	0.1473	0.1575
(*b*) VALUES OF $\frac{Y_1}{r_1}$ (EQUATION (11))								
0.25	0.2101	0.2222	0.2333	0.2436	0.2531	0.2619	0.2701	0.2778
0.23	0.1985	0.2103	0.2212	0.2312	0.2405	0.2491	0.2572	0.2647
0.21	0.1867	0.1983	0.2089	0.2187	0.2277	0.2362	0.2440	0.2513
0.19	0.1748	0.1861	0.1964	0.2060	0.2148	0.2230	0.2307	0.2378
0.17	0.1628	0.1738	0.1838	0.1931	0.2017	0.2097	0.2171	0.2240
0.15	0.1506	0.1612	0.1709	0.1799	0.1883	0.1960	0.2032	0.2099
(*c*) VALUES OF $\frac{G_1}{r_1 l^2_1}$ (EQUATION (14))								
0.25	0.0800	0.0900	0.1000	0.1100	0.1200	0.1300	0.1400	0.1500
0.23	0.0762	0.0859	0.0956	0.1053	0.1150	0.1247	0.1344	0.1441
0.21	0.0725	0.0818	0.0912	0.1006	0.1100	0.1193	0.1287	0.1381
0.19	0.0686	0.0776	0.0867	0.0957	0.1048	0.1138	0.1229	0.1319
0.17	0.0646	0.0733	0.0820	0.0907	0.0994	0.1081	0.1169	0.1256
0.15	0.0606	0.0689	0.0773	0.0856	0.0940	0.1023	0.1107	0.1190
(*d*) VALUES OF $\frac{F_1}{r^2_1 l_1}$ (EQUATION (16))								
0.25	0.0583	0.0666	0.0750	0.0834	0.0916	0.1000	0.1083	0.1167
0.23	0.0533	0.0617	0.0696	0.0775	0.0853	0.0932	0.1012	0.1092
0.21	0.0494	0.0568	0.0643	0.0718	0.0792	0.0866	0.0940	0.1015
0.19	0.0452	0.0522	0.0592	0.0662	0.0732	0.0803	0.0873	0.0943
0.17	0.0410	0.0476	0.0542	0.0608	0.0674	0.0740	0.0806	0.0872
0.15	0.0371	0.0433	0.0495	0.0556	0.0618	0.0680	0.0741	0.0802

Values of g corresponding to N, in Tables 2(*a*), 2(*b*), 2(*c*), and 2(*d*), are as follows (this applies also to Table 4):

N	g	N	g
0.25	1.000	0.19	4.324
0.24	1.347	0.18	5.321
0.23	1.756	0.17	6.536
0.22	2.240	0.16	8.031
0.21	2.814	0.15	9.889
0.20	3.500		

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[10] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 989.

When $g = 1$, the axis is parabolic and Equation (9) becomes:

$$\int y_0 \, dw = \frac{l_1 \, r_1}{E I_c} \left(\frac{1}{3} - \frac{1 - m}{4} \right) \ldots\ldots\ldots\ldots\ldots\ldots (9b)$$

The co-ordinates, X_1 and Y_1, of the center of gravity of the values of dw can now be determined, as follows:

$$X_1 = \frac{\int x \, dw}{\int dw} = \frac{B_1}{W_1} = \frac{(1 + 2m) \, l_1}{3 + 3m} \ldots\ldots\ldots\ldots\ldots (10)$$

and,

$$Y_1 = \frac{\int y_0 \, dw}{\int dw} = \frac{J_1}{W_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The values of X_1 and Y_1 can be computed from Tables 1 and 2(b), respectively. The equation of the elastic curve of vertical displacements is derived as follows: The second derivative of the elastic curve equals,[9]

$$\frac{d^2 \delta}{d z^2} = - \frac{M \, l^2_1}{E \, I_c} \, [1 - (1 - m) \, z]$$

Integrating twice with the proper constants,

$$\delta_{MA} = \frac{l^2_1}{6 \, E \, I_c} \; [- (1 - m) \, z^3 + 3z^2 - (3 + 3m) \, z + (1 + 2m)] \ldots (12)$$

The values of $\frac{E \, I_c}{l^2_1} \, \delta_{MA}$ are given in Table 3(a).

TABLE 3.—ORDINATES OF ELASTIC CURVE OF ONE SEGMENT (a) WITH UNIT MOMENT, AND (b) WITH UNIT VERTICAL FORCE, AT THE CROWN.

(Intermediate values by interpolation)

m	Point 1,* $z = 0.9$	Point 2, $z = 0.8$	Point 3, $z = 0.7$	Point 4, $z = 0.6$	Point 5, (quarter-point); $z = 0.5$	Point 6, $z = 0.4$	Point 7, $z = 0.3$	Point 8, $z = 0.2$	Point 9, $z = 0.1$	Point 10 (crown), $z = 0.0$
(a) VALUE OF $\delta_{MA} \frac{EI_c}{l^2_1}$ (EQUATION (12))										
1.00....	0.0050	0.0200	0.0450	0.0800	0.1250	0.1800	0.2450	0.3200	0.4050	0.5000
0.50....	0.0026	0.0107	0.0247	0.0453	0.0729	0.1080	0.1511	0.2026	0.2633	0.3333
0.25....	0.0013	0.0060	0.0146	0.0280	0.0469	0.0720	0.1041	0.1440	0.1924	0.2500
0.15....	0.0009	0.0041	0.0106	0.0211	0.0365	0.0576	0.0853	0.1205	0.1640	0.2167
(b) VALUE OF $\delta_{VA} \frac{EI_c}{l^3_1}$ (EQUATION (15))										
1.00....	0.0048	0.0187	0.0405	0.0693	0.1042	0.1440	0.1878	0.2347	0.2835	0.3333
0.50....	0.0025	0.0099	0.0222	0.0389	0.0599	0.0846	0.1125	0.1429	0.1752	0.2083
0.25....	0.0013	0.0056	0.0130	0.0237	0.0378	0.0549	0.0748	0.0971	0.1210	0.1458
0.15....	0.0009	0.0038	0.0093	0.0177	0.0289	0.0430	0.0598	0.0788	0.0993	0.1208

* For $z = 1.0$ (at the springing line, Point 0), $\delta_{MA} \frac{EI_c}{l^2_1}$ and $\delta_{VA} \frac{EI_c}{l^3_1} = 0$ at all values of m.

The displacements produced by a unit vertical load will now be determined. The vertical displacement at C (Fig. 4), is:

$$\int M\,x\,dw = \int x^2\,dw = \frac{l^3_1}{E\,I_c}\int (z^2 - z^3 + m\,z^3)\,dz$$

from which,

$$\int x^2\,dw = \frac{l^3_1}{E\,I_c}\,\frac{1+3\,m}{12} = \frac{D_1}{E\,I_c} \quad \dots\dots\dots\dots\dots (13)$$

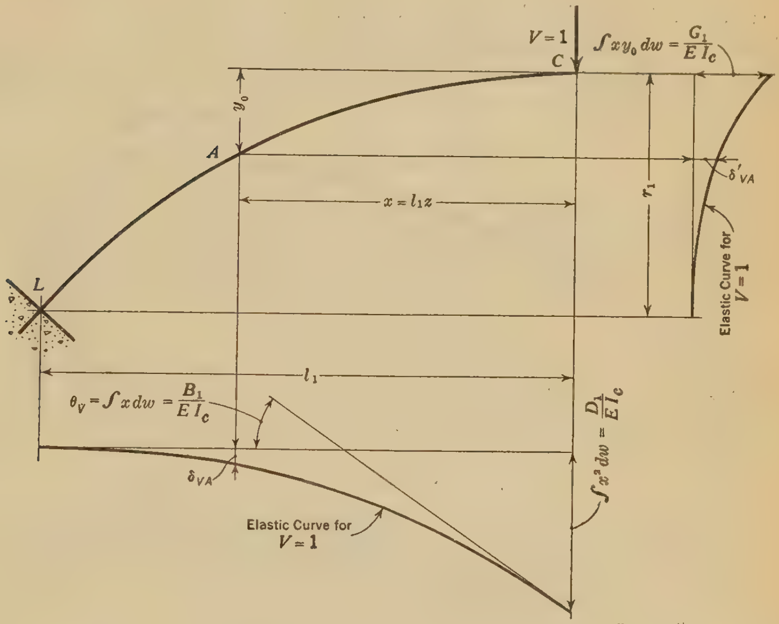

$V=1$
C
$\int x y_0\,dw = \frac{G_1}{E\,I_c}$
y_0
A
δ'_{VA}
$x = l_1 z$
r_1
Elastic Curve for $V=1$
L
l_1
$\theta_V = \int x\,dw = \frac{B_1}{E\,I_c}$
δ_{VA}
Elastic Curve for $V=1$
$\int x^2\,dw = \frac{D_1}{E\,I_c}$

FIG. 4.

in which, $D_1 = E\,I_c \int x^2\,dw$ can be computed from Table 1.

The horizontal displacement at C due to a unit vertical load at C is:

$$\int M\,y_0\,dw = \int x\,y_0\,dw = \frac{r_1\,l^2_1}{E\,I_c\,(g-1)}\left[-\frac{g-1}{k^2} - \frac{1}{2} + \frac{1-m}{3} + \frac{2\,(1-m)}{k^2}\left(g - \frac{\sqrt{g^2-1}}{k}\right) + \frac{m\,\sqrt{g^2-1}}{k}\right] \dots\dots\dots (14)$$

or,

$$\int x\,y_0\,dw = \frac{G_1}{E\,I_c} \quad \dots\dots\dots\dots\dots\dots\dots\dots (14a)$$

in which, $G_1 = E\,I_c \int x\,y_0\,dw$, and can be obtained from Table 2(c). For the special case of the parabolic rib when $g = 1$, a simple formula can be derived using the relation, $y_0 = r_1 z^2$. Then,

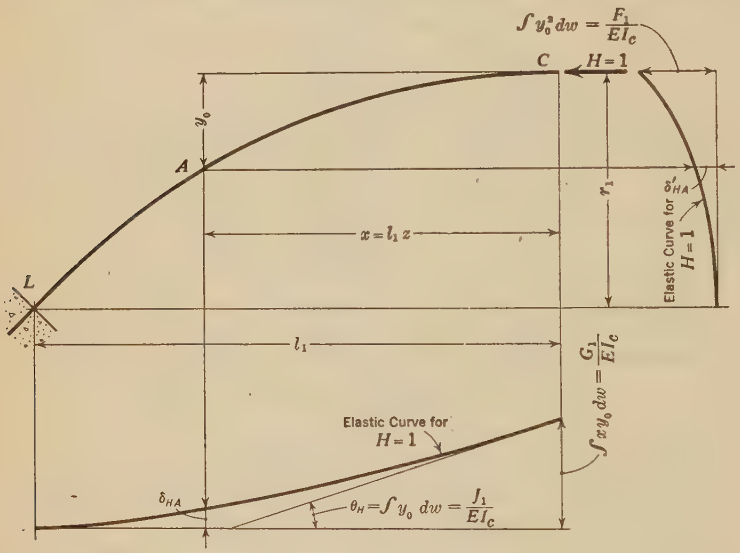
$$\int x\, y_0\, dw = \frac{r_1\, l^2_1}{E\, I_c} \left(\frac{1}{4} - \frac{1 - m}{5} \right) \ldots\ldots\ldots\ldots\ldots (14b)$$

The equation of the elastic curve of vertical displacements due to the unit vertical load at C is obtained in the same manner as Equation (12) and is:

$$\delta_{VA} = \frac{l^3_1}{12\, E\, I_c} \left[-(1 - m)\, z^4 + 2\, z^3 - (2 + 4\, m)\, z + (1 + 3\, m) \right] \ldots (15)$$

The values of δ_{VA} can be obtained from Table 3(b).

The displacements produced by a unit horizontal load will be as follows (see Fig. 5): The vertical displacement of C due to a unit horizontal load at C is equal to the horizontal displacement due to a vertical load and is given by Equations (14) and (14a), with Table 2(c). The horizontal displacement at C due to a unit horizontal load at C is:



$\int y_0^2\, dw = \frac{F_1}{EI_c}$
C
H=1
y_0
A
r_1
δ'_{HA}
Elastic Curve for H = 1
$x = l_1 z$
L
l_1
$\int x y_0\, dw = \frac{G_1}{EI_c}$
Elastic Curve for H = 1
δ_{HA}
$\theta_H = \int y_0\, dw = \frac{J_1}{EI_c}$

FIG. 5.

$$\int y^2_0\, dw = Y^2_1 \int dw + \int y^2\, dw$$

in which, y is the ordinate from the axis through the center of gravity of the values of dw.[11]

The horizontal displacement is given by the formula,

$$\int y^2_0\, dw = \frac{F_1}{E\, I_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The value of F_1, can be taken from Table 2(d).

---

[11] The values of $\int y^2\, dw$ are given in *Transactions*, Am. Soc. C. E., Vol. 88 (1925), Table 18, p 991.

The equation of the elastic curve of vertical displacements due to the unit horizontal load at C is derived from:

$$\frac{d^2\,\delta_{HA}}{dz^2} = \frac{-\,l^2_1\,r_1}{E\,I_c\,(g-1)}\,(\cosh zk - 1)\,[1 - (1 - m)\,z]$$

The values of δ_{HA} are given in Table 4.

TABLE 4.—ORDINATES OF ELASTIC CURVE OF ONE SEGMENT WITH UNIT HORIZONTAL FORCE AT CROWN; $\left(\text{VALUES OF } \delta_{HA}\,\dfrac{E\,I_c}{l^2_1\,r_1}\right)$.

(Intermediate values by interpolation)

N*	Point 1,† $z = 0.9$	Point 2, $z = 0.8$	Point 3, $z = 0.7$	Point 4, $z = 0.6$	Point 5 (quarter-point), $z = 0.5$	Point 6, $z = 0.4$	Point 7, $z = 0.3$	Point 8, $z = 0.2$	Point 9, $z = 0.1$	Point 10 (crown), $z = 0.0$
					$m = 0.15$					
0.25....	0.0008	0.0036	0.0083	0.0150	0.0235	0.0333	0.0442	0.0558	0.0679	0.0800
0.23....	0.0008	0.0035	0.0081	0.0146	0.0227	0.0322	0.0426	0.0536	0.0650	0.0764
0.21....	0.0008	0.0035	0.0080	0.0142	0.0220	0.0310	0.0407	0.0512	0.0618	0.0726
0.19....	0.0008	0.0033	0.0078	0.0137	0.0212	0.0297	0.0389	0.0486	0.0586	0.0687
0.17....	0.0008	0.0033	0.0075	0.0132	0.0202	0.0282	0.0369	0.0459	0.0552	0.0646
0.15....	0.0008	0.0032	0.0073	0.0128	0.0194	0.0269	0.0349	0.0433	0.0520	0.0606
					$m = 0.20$					
0.25....	0.0010	0.0043	0.0099	0.0176	0.0273	0.0384	0.0505	0.0634	0.0766	0.0899
0.23....	0.0010	0.0043	0.0097	0.0173	0.0265	0.0371	0.0487	0.0609	0.0734	0.0860
0.21....	0.0010	0.0042	0.0095	0.0168	0.0256	0.0356	0.0466	0.0581	0.0699	0.0819
0.19....	0.0010	0.0041	0.0092	0.0163	0.0247	0.0343	0.0445	0.0554	0.0666	0.0777
0.17....	0.0010	0.0041	0.0091	0.0157	0.0236	0.0327	0.0424	0.0525	0.0629	0.0733
0.15....	0.0010	0.0040	0.0088	0.0151	0.0227	0.0311	0.0403	0.0496	0.0592	0.0689
					$m = 0.25$					
0.25....	0.0013	0.0052	0.0116	0.0204	0.0311	0.0434	0.0569	0.0710	0.0855	0.1000
0.23....	0.0013	0.0051	0.0114	0.0199	0.0302	0.0419	0.0547	0.0682	0.0819	0.0957
0.21....	0.0013	0.0051	0.0111	0.0193	0.0292	0.0404	0.0525	0.0652	0.0782	0.0912
0.19....	0.0013	0.0050	0.0109	0.0188	0.0282	0.0388	0.0503	0.0623	0.0745	0.0867
0.17....	0.0012	0.0048	0.0106	0.0181	0.0271	0.0371	0.0479	0.0591	0.0706	0.0821
0.15....	0.0012	0.0047	0.0103	0.0175	0.0260	0.0355	0.0455	0.0559	0.0666	0.0772
					$m = 0.30$					
0.25....	0.0015	0.0060	0.0133	0.0231	0.0350	0.0485	0.0631	0.0784	0.0942	0.1101
0.23....	0.0015	0.0059	0.0131	0.0225	0.0339	0.0469	0.0608	0.0754	0.0904	0.1054
0.21....	0.0014	0.0058	0.0128	0.0219	0.0329	0.0452	0.0585	0.0724	0.0865	0.1006
0.19....	0.0014	0.0057	0.0124	0.0213	0.0317	0.0434	0.0560	0.0690	0.0823	0.0957
0.17....	0.0014	0.0056	0.0121	0.0206	0.0306	0.0416	0.0534	0.0657	0.0782	0.0907
0.15....	0.0014	0.0055	0.0118	0.0200	0.0294	0.0397	0.0508	0.0623	0.0739	0.0856
					$m = 0.40$					
0.25....	0.0019	0.0076	0.0166	0.0285	0.0426	0.0585	0.0756	0.0936	0.1117	0.1300
0.23....	0.0019	0.0075	0.0164	0.0278	0.0414	0.0566	0.0730	0.0900	0.1073	0.1247
0.21....	0.0019	0.0074	0.0159	0.0270	0.0401	0.0546	0.0701	0.0864	0.1028	0.1193
0.19....	0.0019	0.0073	0.0156	0.0264	0.0388	0.0527	0.0675	0.0829	0.0983	0.1139
0.17....	0.0019	0.0071	0.0152	0.0255	0.0374	0.0505	0.0644	0.0789	0.0935	0.1081
0.15....	0.0019	0.0070	0.0149	0.0247	0.0360	0.0484	0.0615	0.0750	0.0886	0.1023
					$m = 0.50$					
0.25....	0.0024	0.0093	0.0200	0.0339	0.0503	0.0685	0.0881	0.1085	0.1291	0.1500
0.23....	0.0024	0.0091	0.0196	0.0331	0.0489	0.0665	0.0852	0.1046	0.1244	0.1442
0.21....	0.0024	0.0090	0.0191	0.0322	0.0474	0.0641	0.0820	0.1004	0.1192	0.1380
0.19....	0.0023	0.0088	0.0188	0.0314	0.0459	0.0619	0.0788	0.0964	0.1141	0.1320
0.17....	0.0023	0.0087	0.0183	0.0304	0.0443	0.0595	0.0755	0.0920	0.1088	0.1256
0.15....	0.0023	0.0085	0.0178	0.0294	0.0426	0.0570	0.0720	0.0875	0.1033	0.1190

* Corresponding values of g previously listed.

† For $z = 1.0$ (that is, at the springing line, Point 0), $\delta_{HA}\,\dfrac{E\,I_c}{l^2_1\,r_1} = 0$ at all values of N and m.

II.—REACTIONS OF UNSYMMETRICAL ARCH

Geometry of Unsymmetrical Rib.—Assume two rib segments of different proportions joined at the crown. In the formulas the principal dimensions of the left segment are indicated with a subscript, 1, and those of the right segment with a subscript, 2. Such terms as g, m, d_S, etc., refer to their respective segments without special designation. The crown thickness, d_C, is the same for both segments.

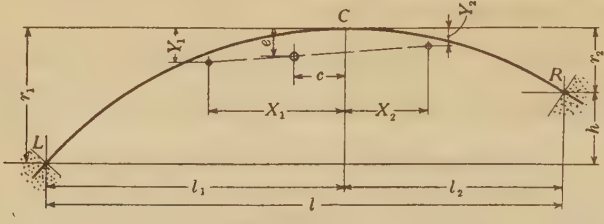


FIG. 6.

The origin of co-ordinates will be taken at the crown point and the co-ordinates of the center of gravity of the values of dw for the entire rib may be determined by taking moments. Referring to Fig. 6.

$$c = \frac{X_1 W_1 - X_2 W_2}{W_1 + W_2} = \frac{B_1 - B_2}{W_1 + W_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

and,

$$e = \frac{Y_1 W_1 + Y_2 W_2}{W_1 + W_2} = \frac{J_1 + J_2}{W_1 + W_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

The entire rib will now be considered as a cantilever free at the left end and fixed at the right end, with the reactions, H, V, and M, acting on the left end of the rib through weightless levers. Any reaction that passes through the center of gravity of the values of dw, the co-ordinates of which are c and e, will produce no rotation of the left end, L. By writing equations for the vertical and horizontal displacements of L, the H and V reactions of the fixed arch can be derived.

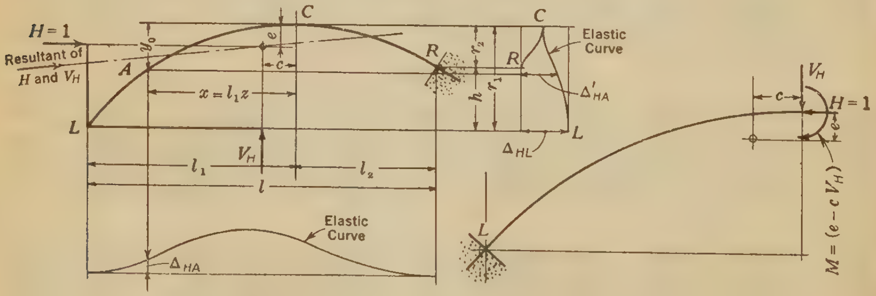


FIG. 7. FIG. 8.

DETERMINATION OF HORIZONTAL REACTION

Assume the arch rib as in Fig. 7 to be acted upon by a unit value of H with a vertical reaction, V_H, of sufficient magnitude to cause the left end, L, to move in a horizontal direction. In the case of the symmetrical arch, the value of V_H is zero. The elastic curve of vertical displacements of the rib, Δ_{HA}, produced by H and V_H, when divided by the corresponding horizontal displacement, Δ_{HL}, of the left end, L, is the influence line of horizontal reactions when the rib is fixed at both ends and acted upon by a single vertical load.

In order to determine the value of V_H and the deformations of the rib, the elastic properties given in Part I will be used. Since $H = 1$, the bending moment at any point in the rib is:

$$M = V_H\,(x + c) + y_0 - e$$

assuming, c to be positive, and x to be positive to the right and negative to the left of the crown point. The vertical displacement of the crown point, C, in relation to L, due to the bending in the left segment, will then be:

$$\int_c^L M\,x\,dw = V_H\left(\int x^2\,dw + c\int x\,dw\right) + \int x\,y_0\,dw - e\int x\,dw$$

or,

$$E\,I_c\int_c^L M\,x\,dw = V_H\,(-D_1 + c\,B_1) + G_1 - e\,B_1$$

for the right segment the displacement of C relative to R is:

$$E\,I_c\int_R^c M\,x\,dw = V_H\,(D_2 + c\,B_2) + G_2 - e\,B_2$$

Since the vertical displacement of L relative to R is zero, it follows that:

$$E\,I_c\int_R^L M\,x\,dw = V_H\,(-\,D_1 - D_2 + cB_1 - cB_2) + G_1 - G_2$$
$$-\,eB_1 + eB_2 = 0$$

from which,

$$V_H = \frac{G_1 - G_2 - e\,(B_1 - B_2)}{D_1 + D_2 - c\,(B_1 - B_2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

It should be noted that V_H is the tangent of the angle between the horizontal and the reaction produced when the abutment of the arch is displaced horizontally without rotation.

It is now possible to determine the displacements, Δ_{HL} and Δ_{HA}, shown in Fig. 7. As before, $M = V_H\,(x + c) + y_0 - e$. The horizontal displacement of the crown point, C, relative to L is:

$$\int M\,y_0\,dw = V_H\int (x\,y_0 + c\,y_0)\,dw + \int y_0^2\,dw - e\int y_0\,dw$$

or,

$$E\,I_c\int M\,y_0\,dw = -\,V_H\,(G_1 - c\,J_1) + F_1 - e\,J_1$$

For the right segment the horizontal displacement of C relative to R is:

$$E\,I_c \int M\,y_0\,dw = V_H\,(G_2 + c\,J_2) + F_2 - e\,J_2$$

The total displacement is then equal to the sum of the displacements for the two sides, hence,

$$\Delta_{HL}\,E\,I_C = F_1 + F_2 - e\,(J_1 + J_2) - V_H\,[(G_1 - G_2) - c\,(J_1 + J_2)] \,..(20)$$

Since $e\,(B_1 - B_2) = c\,(J_1 + J_2)$, Equation (20) can be written:

$$\Delta_{HL}\,E\,I_C = F_1 + F_2 - e\,(J_1 + J_2) - V_H\,[G_1 - G_2 - e\,(B_1 - B_2)] \,..(20a)$$

In order to obtain the elastic curve of the rib produced by $H = 1$, consider the two segments again separated as in Part I. The forces acting on the left-hand segment are then shown in Fig. 8. The vertical displacement of any point, A, is then equal to:

For the left-hand segment:

$$\Delta_{HA} = \delta_{HA} - V_H\,\delta_{VA} - (e - c\,V_H)\,\delta_{MA} \,............(21L)$$

For the right-hand segment:

$$\Delta_{HA} = \delta_{HA} + V_H\,\delta_{VA} - (e - c\,V_H)\,\delta_{MA} \,............(21R)$$

and the horizontal reaction due to a vertical load, P, at the point, A, is:

$$H = P\,\frac{\Delta_{HA}}{\Delta_{HL}} \,........................(22)$$

All the values in these formulas can be obtained from the tables. The values of δ are, of course, not the same for the two segments.

Determination of Vertical Reaction

Assume a unit vertical force, $V = 1$, acting on the left end of the arch rib and passing through the center of gravity of the values of dw as shown in Fig. 9. In order to make the left end move vertically it will be necessary to apply a horizontal force, H_V, also acting through the center of gravity, so that there will be no rotation at L. If the arch were symmetrical, H_V would be zero.

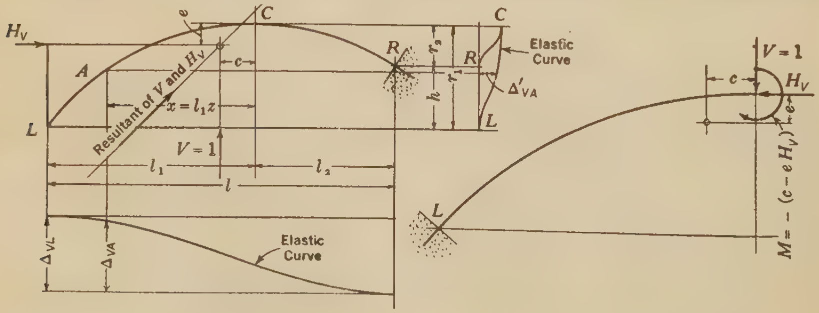


FIG. 9.

FIG. 10.

The magnitude of H_V can be obtained by writing the equation for the horizontal displacement of L and making it equal to zero. This can be done as before in determining V_H, thus:

$$M = (x + c) + H_V (y_0 - e)$$

Following the procedure used in the derivation of Equation (20) the horizontal displacement of L will be:

$$E I_c \int M y_0 \, dw = H_V [F_1 + F_2 - e (J_1 + J_2)] - (G_1 - G_2) + c (J_1 + J_2)$$

or,

$$H_V = \frac{G_1 - G_2 - c (J_1 + J_2)}{F_1 + F_2 - e (J_1 + J_2)} = \frac{G_1 - G_2 - e (B_1 - B_2)}{F_1 + F_2 - e (J_1 + J_2)} \quad \ldots\ldots (23)$$

The value of H_V is also the tangent of the angle between the vertical and the resultant when the abutment is displaced vertically without rotation.

The influence line for vertical reaction can now be found in the same manner as the horizontal reaction. The vertical displacement of the left end produced by $V = 1$ will be, from the value of $\int M \, x \, dw$ used in the derivation of Equation (19) with the proper values of V and H_V substituted:

$$\Delta_{VL} \, E I_c = D_1 + D_2 - c (B_1 - B_2) - H_V [G_1 - G_2 - e (B_1 - B_2)] \ldots (24)$$

The elastic curve of vertical displacements is obtained as for H. For the left segment the vertical displacement at any point, A (Fig. 10), is:

$$\Delta_{VA} = \Delta_{VL} - \delta_{VA} + H_V \, \delta_{HA} + (c - e H_V) \, \delta_{MA} \ldots\ldots\ldots (25L)$$

and for the right segment:

$$\Delta_{VA} = \delta_{VA} + H_V \, \delta_{HA} + (c - e H_V) \, \delta_{MA} \ldots\ldots\ldots\ldots (25R)$$

The vertical reaction due to a vertical load, P, at A is, then:

$$V = P \frac{\Delta_{VA}}{\Delta_{VL}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

DETERMINATION OF MOMENT AT SPRINGING

In order to determine the moment at the springing, the rib will be considered fixed at the right end with a unit moment acting at the left end. The elastic line of vertical displacements caused by this moment will be the influence line for a moment, M', which, added to the moments produced by H and V, gives the moment at the left end of the fixed arch.

The vertical displacement, Δ_{MA}, at any point in the right-hand segment is given by Equation (12):

$$\Delta_{MA} \, E I_c = \delta_{MA} \, E I_c = \frac{l_2^2}{6} [(1 - m) z^3 - 3 z^2 + (3 + 3 m) z - (1 + 2m)] \ldots (27R)$$

Values of $\Delta_{MA} \, E I_c$ are given in Table 3(a).

For the left segment the ordinate, Δ_{MA}, of the elastic curve is made up of two parts, a and b. From Fig. 11 it will be seen that,

$$\theta_{ML} = \int dw = \frac{1}{EI_c}\,(W_1 + W_2)$$

and,

$$b = \theta_{ML}x - \frac{B_1 - B_2}{EI_c} = \frac{1}{EI_c}\,[l_1\,(W_1 + W_2)\,z - (B_1 - B_2)]$$

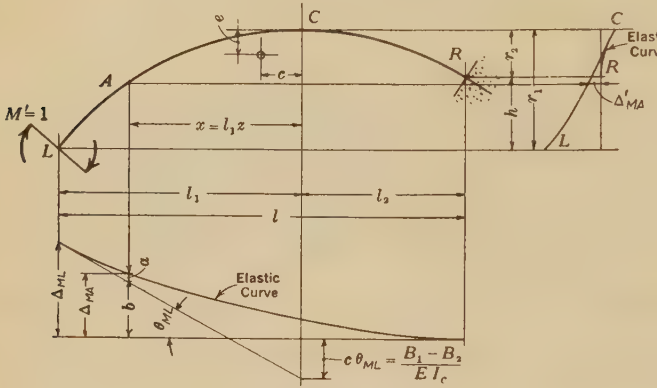


FIG. 11.

The value of a is given by Equation (12); thus:

$$a\,E\,I_c = \delta_{MA}\,E\,I_c = \frac{l_1^2}{6}\,[(1 - m)z^3 - 3z^2 + (3 + 3m)z - (1 + 2m)]$$

Then,

$$\Delta_{MA}\,E\,I_c = (a + b)\,E\,I_c = l_1\,(W_1 + W_2)z - (B_1 - B_2)$$
$$+ \frac{l_1^2}{6}\,[(1 - m)z^3 - 3z^2 + (3 + 3m)z - (1 + 2m)] \ldots\ldots (27L)$$

The vertical displacement at the left end will be equal to,

$$\Delta_{ML} = (l_1 - c)\theta_{ML} = (l_1 - c)\;(W_1 + W_2)\;\frac{1}{EI_c}$$

The value of M' is,

$$M' = \frac{\Delta_{MA}}{\theta_{ML}} = \frac{\Delta_{MA}\,EI_c}{W_1 + W_2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

The moment at the left springing of the fixed arch produced by a concentrated vertical load is:

$$M = M' + H\,(r_1 - e) - V\,(l_1 - c) \ldots\ldots\ldots\ldots\ldots (29)$$

EFFECT OF HORIZONTAL LOADS

The effect of horizontal loads has not been treated in detail because in practical design it is seldom necessary to consider it. However, if it is desired to do so, the reactions produced by a horizontal load at any point can be determined by substituting the values of Δ'_{HA}, Δ'_{VA}, and Δ'_{MA}, for Δ_{HA}, Δ_{VA}, and Δ_{MA}, respectively, in Equations (22), (26), and (28) (see Figs. 7, 9, and 11).

EFFECT OF TEMPERATURE CHANGE

If the left end of the arch rib were free, the effect of a change in temperature would be to lengthen or to shorten the chord of the arch, by an amount equal to $t\,\alpha\,\sqrt{l^2+h^2}$, producing a corresponding displacement of the left end. Since the left end is actually fixed, the reactions produced by the temperature change will be those required to return the end to its original position.

Considering the horizontal component of the temperature displacement, $t\,\alpha\,l$, only: $H = \frac{t\,\alpha\,l}{\Delta_{HL}}$, and $V = V_H \frac{t\,\alpha\,l}{\Delta_{HL}}$. Considering the vertical component, $t\,\alpha\,h$, only: $V = \frac{t\,\alpha\,h}{\Delta_{VL}}$, and $H = H_V \frac{t\,\alpha\,h}{\Delta_{VL}}$. Then, the reactions at L, due to temperature change, are:

$$H_t = \frac{t\,\alpha\,l}{\Delta_{HL}} + H_V \frac{t\,\alpha\,h}{\Delta_{VL}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

$$V_t = \frac{t\,\alpha\,h}{\Delta_{VL}} + V_H \frac{t\,\alpha\,l}{\Delta_{HL}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

and,

$$M_t = H_t\,(r_1 - e) - V_t\,(l_1 - c) \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

EFFECT OF RIB-SHORTENING OR ABUTMENT YIELDING

Let Δx represent the horizontal component of rib-shortening due to full load on the arch. Then,

$$\Delta x = -H \int_L^R \frac{dx}{\cos\phi\, E\, A} = -H\left(\frac{l_1}{E\,A'_m} + \frac{l_2}{E\,A'_m}\right) \ldots\ldots (33)$$

in which, $A'_m = \dfrac{l_1}{\int_L^c \frac{dx}{A\cos\phi}}$, or $\dfrac{l_2}{\int_R^c \frac{dx}{A\cos\phi}} = A_c\,C'_m$; A_c is the area of cross-section of the rib at the crown; and C'_m is a coefficient given by Fig. 12. The value of A'_m must be computed from each segment of rib separately. The vertical component of rib-shortening is:

$$\Delta y = -H \int_L^R \frac{dy}{\cos\phi\, E\, A} = -H\left(\frac{r_1}{E\,A''_m} - \frac{r_2}{E\,A''_m}\right) \ldots\ldots (34)$$

in which, $A''_m = \dfrac{r_1}{\int_L^c \frac{dy}{A\cos\phi}}$, or $\dfrac{r_2}{\int_R^c \frac{dy}{A\cos\phi}} = A_c\,C''_m$; C''_m is given by Fig. 12 and, as before, A''_m must be computed for each segment of rib. These values of Δx and Δy can be substituted in Equations (30) and (31) for the values of $t\,\alpha\,l$ and $t\,\alpha\,h$, respectively, and the reactions at L due to rib-shortening are:

$$H_{RS} = \frac{\Delta x}{\Delta_{HL}} + H_V \frac{\Delta y}{\Delta_{VL}} \tag{35}$$

$$V_{RS} = \frac{\Delta y}{\Delta_{VL}} + V_H \frac{\Delta x}{\Delta_{HL}} \tag{36}$$

and,

$$M_{RS} = H_{RS}\,(r_1 - e) - V_{RS}\,(l_1 - c) \tag{37}$$

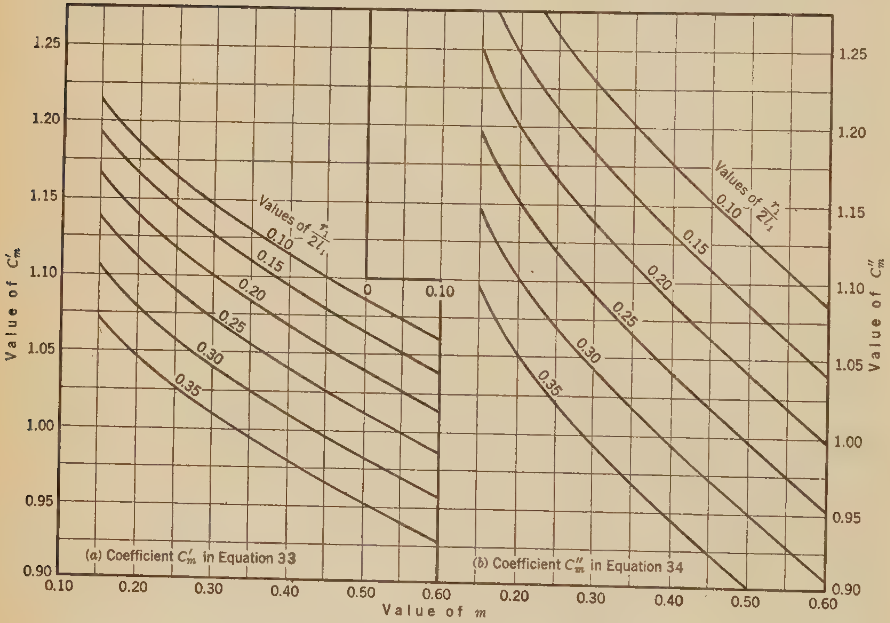


FIG. 12.—AVERAGE CROSS-SECTION FOR COMPUTING RIB-SHORTENING.

Equations (35), (36), and (37) can also be used to determine the reactions produced by a horizontal or vertical yielding of the abutments equal to Δx, or Δy, respectively. The reactions at the left end produced by a clockwise rotation of the left abutment equal to θ will be:

$$M' = \frac{\theta\,E I_c}{W_1 + W_2} \tag{38}$$

$$H_L = \frac{\theta\,(r_1 - e)}{\Delta_{HL}} - H_V \frac{\theta\,(l_1 - c)}{\Delta_{VL}} \tag{39}$$

$$V_L = -\frac{\theta\,(l_1 - c)}{\Delta_{VL}} + V_H\,\frac{\theta\,(r_1 - e)}{\Delta_{HL}} \quad \text{.............}(40)$$

and,

$$M_L = M' + H_L\,(r_1 - e) - V_L\,(l_1 - c) \quad \text{.............}(41)$$

III.—EXAMPLE

Tables and Formulas.—The integrals of functions of the elastic weight, *dw*, for each segment of the arch rib, are designated as follows:

$$W = E\,I_c \int dw \text{ (see Table 1)}$$

$$B = E\,I_c \int x\,dw \text{ (see Table 1)}$$

$$D = E\,I_c \int x^2\,dw \text{ (see Table 1)}$$

$$J = E\,I_c \int y_0\,dw \text{ (see Table 2}(a))$$

$$G = E\,I_c \int x\,y_0\,dw \text{ (see Table 2}(c))$$

$$F = E\,I_c \int y^2_0\,dw \text{ (see Table 2 }(d))$$

Since the arch rib is unsymmetrical the values of d_s and I_s may not be the same for the two ends of the rib. The values of N, ϕ_s, m, k, and g will be different for the two segments of the rib to the left and right of the crown point.

In the formulas and tables of this paper the subscript, 1, is used to indicate the left-hand segment of the arch. The formulas can be applied to the right-hand segment by changing the subscript, 1, to 2 and using the proper values of N, ϕ_s, m, k, or g, except those formulas which apply to only one segment and are marked L or R.

Arch-axis co-ordinates are computed by Equation (1), in which values of $\frac{y_0}{r}$ can be picked from Table 6 of the writer's previous paper.[12] Likewise, the dead load reactions are computed by Equations (2) and (3), using Table 9 of the previous paper.[13]

Numerical Example.—Consider an arch of the dimensions given in Fig. 13 with the following dead loads per linear foot: $w_c = 520$ lb; $w_{s1} = 1\,030$ lb; and $w_{s2} = 1\,000$ lb. Then, $g_1 = \frac{w_{s1}}{w_c} = 1.981$; $g_2 = \frac{w_{s2}}{w_c} = 1.923$.

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[12] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 973.

[13] *Loc. cit.*, p. 976.

The crown point is located in the following manner: Taking the values of C_d from Table 9 of the previous paper.[13]

$$H_{d1} = \frac{4\,C_{d1}\,w_c\,l^2_1}{r_1} = H_{d2} = \frac{4\,C_{d2}\,w_c\,l^2_2}{r_2}$$

$$l_1 = l_2\sqrt{\frac{r_1}{r_2}\left(\frac{C_{d2}}{C_{d1}}\right)} = 1.1503\,l_2$$

Since $l_1 + l_2 = 140$ ft; and, $l_2 = \dfrac{140}{2.1503} = 65.11$ ft; $l_1 = 140 - 65.11 = 74.89$ ft; $H_{d1} = 41\,938$ lb; and $H_{d2} = 41\,943$ lb.

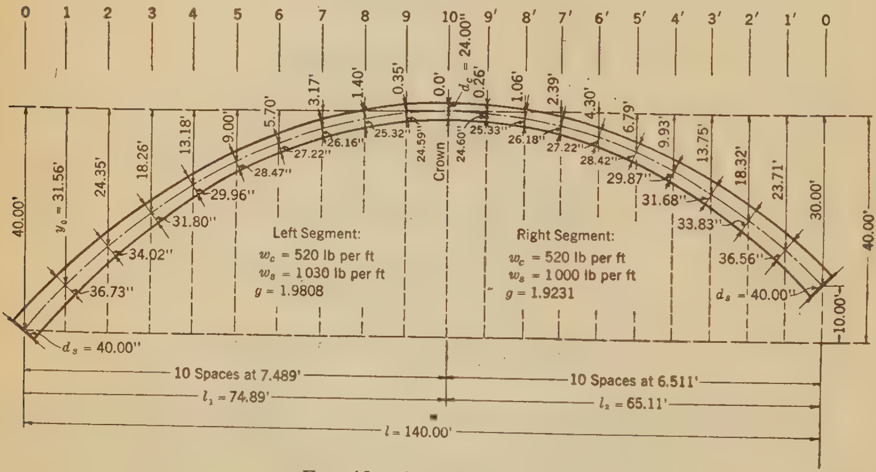
0
1
2
3
4
5
6
7
8
9
10
9′
8′
7′
6′
5′
4′
3′
2′
1′
0
d_c = 24.00"
40.00'
y_0 = 31.56'
24.35'
18.26'
13.18'
9.00'
5.70'
3.17'
1.40'
0.35'
0.0'
0.26'
1.06'
2.39'
4.30'
6.79'
9.93'
13.75'
18.32'
23.71'
30.00'
40.00'
10.00'
36.73"
34.02"
31.80"
29.96"
28.47"
27.22"
26.16"
25.32"
24.59"
Crown
24.60"
25.33"
26.18"
27.22"
28.42"
29.87"
31.68"
33.83"
36.56"
d_s = 40.00"
d_s = 40.00"
Left Segment:
w_c = 520 lb per ft
w_s = 1030 lb per ft
g = 1.9808
Right Segment:
w_c = 520 lb per ft
w_s = 1000 lb per ft
g = 1.9231
10 Spaces at 7.489'
l_1 = 74.89'
10 Spaces at 6.511'
l_2 = 65.11'
l = 140.00'

FIG. 13.—ARCH DIMENSIONS

The values of tan ϕ_s, m, and N may now be computed (see Table 9 of the previous paper.[13])

For the left segment:

$$\tan\phi_{s1} = \frac{4.553 \times r_1}{2\,l_1} = 1.21592;\ \phi_{s1} = 50°\ 33'\ 50''$$

$$\cos\phi_{s1} = 0.63522;\ \text{and}\ m_1 = \frac{d^3_c}{d^3_s\cos\phi_{s1}} = 0.34004$$

Interpolating from Table 6 of the previous paper[12]: $N_1 = 0.22515$; and y_0 (Point 5) $= N_1\,r_1 = 9.006$ ft.

For the right segment:

$$\tan\phi_{s2} = 1.04224;\ \phi_{s2} = 46°\ 11'\ 04'';\ m_2 = 0.31198$$

$$N_2 = 0.2264;\ \text{and}\ y_0\ \text{(Point 5)} = 6.792\ \text{ft.}$$

The rib-thickness values shown in Fig. 13 were computed with Equations (5) and (5a), using Tables 12 and 13 of the previous paper.[14]

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[14] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 980.

The following values are computed by taking the values from the tables as noted and multiplying by the corresponding values of l and r:

$W_1 =$ 50.177; $W_2 =$ 42.713 from Table 1
$B_1 =$ 1 570.0; $B_2 =$ 1 148 from Table 1
$J_1 =$ 473.3; $J_2 =$ 296.51 from Table 2(a)
$X_1 =$ 31.30 ft; $X_2 =$ 26.87 ft. from Table 1
$Y_1 =$ 9.43 ft; $Y_2 =$ 6.94 ft. from Table 2(b)
$D_1 =$ 70 690; $D_2 =$ 44 520 from Table 1
$G_1 =$ 25 100; $G_2 =$ 13 600 from Table 2(c)
$F_1 =$ 9 874; $F_2 =$ 4 588 from Table 2(d)

Co-ordinates of the center of gravity of the elastic weights, dw, for the entire rib are: $c = 4.524$ ft (Equation (17)); and $e = 8.287$ ft (Equation (18)).

The displacements of the left end resulting from unit reactions at the left end, with the entire rib acting as a cantilever supported at the right end, are computed as follows: $V_H = 0.07064$ (Equation (19)); $H_V = 0.9901$ (Equation (23)); $\Delta_{HL}\ EI_c = 7\ 438$ (Equation (20)); and $\Delta_{VL}\ EI_c = 105\ 376$ (Equation (24)).

Table 5 shows the computations of the displacements, δ, of Points A resulting from unit forces and moments at the crown of each segment acting as a cantilever supported at the springing.

TABLE 5.—VALUES OF δ_{HA}, δ_{VA}, AND δ_{MA}. DISPLACEMENTS OF CANTILEVER SEGMENTS DUE TO UNIT LOAD AT CROWN

Point*	z	$\frac{\delta_{HA}\ EI_c}{l^2 r_1}$ (From Table 4)	$\delta_{HA}\ EI_c$	$\frac{\delta_{VA}\ EI_c}{l_1^3}$ (From Table 3 (b))	$\delta_{VA}\ EI_c$	$\frac{\delta_{MA}\ EI_c}{l_1^2}$ (From Table 3 (a))	$\delta_{MA}\ EI_c$
(a) LEFT SEGMENT, WITH $l_1 = 74.89$ FEET; $r_1 = 40.00$ FEET; $g = 1.981$; $N = 0.225$; AND $m = 0.340$.							
2	0.8	0.0065	1 458	0.0071	2 982	0.0077	43.2
4	0.6	0.0245	5 496	0.0292	12 265	0.0342	191.8
6	0.4	0.0503	11 284	0.0656	27 553	0.0850	476.7
8	0.2	0.0804	18 037	0.1136	47 714	0.1651	926.0
10	0.0	0.1119	25 104	0.1683	70 690	0.2800	1 570.4
(b) RIGHT SEGMENT, WITH $l_2 = 65.11$ FEET; $r_2 = 30.00$ FEET; $g = 1.923$; $N = 0.226$; AND $m = 0.312$.							
2	0.8	0.0061	776	0.0066	1 822	0.0071	30.1
4	0.6	0.0230	2 925	0.0275	7 591	0.0323	136.9
6	0.4	0.0477	6 066	0.0622	17 169	0.0809	343.0
8	0.2	0.0765	9 729	0.1084	29 921	0.1585	671.9
10	0.0	0.1069	13 595	0.1613	44 522	0.2707	1 147.6

* For Points 0 all values of $\delta = 0$

Table 6 shows the method of computing: (1) The total displacements, Δ, of Points A, resulting from the unit reactions at the left end with the entire rib acting as a cantilever supported at the right end; (2) the reactions at the left end resulting from unit loads at Points A with the rib fixed at both ends. The reactions at any other point may then be computed by statics.

TABLE 6.—VALUES OF REACTIONS; H, V, AND M, AT LEFT END OF ARCH, DUE TO UNIT LOADS

Point*	z	$\Delta_{HA} EI_c$ Equation (21)	$H = \frac{\Delta_{HA}}{\Delta_{HL}}$	$\Delta_{VA} EI_c$ Equation (25)	$V = \frac{\Delta_{VA}}{\Delta_{VL}}$	$\Delta_{MA} EI_c$ Equation (27)	M' Equation (28)	M_L Equation (29)
				(a) LEFT SEGMENT.				
2	0.8	903	0.1214	103 679	0.9839	5 186	55.83	—9.55
4	0.6	3 102	0.4170	97 847	0.9286	3 943	42.45	—9.67
6	0.4	5 540	0.7448	87 240	0.8279	2 838	30.55	—4.09
8	0.2	7 289	0.9800	72 111	0.6843	1 896	20.41	+3.34
10	0.0	7 600	1.0216	53 760	0.5102	1 148	12.36	+8.86
				(b) RIGHT SEGMENT				
2	0.8	665	0.0894	2 479	0.0235	30.1	0.32	+1.51
4	0.6	2 369	0.3185	9 983	0.0947	136.9	1.47	+4.91
6	0.4	4 546	0.6112	21 912	0.2079	343.0	3.69	+8.44
8	0.2	6 490	0.8725	36 981	0.3510	671.9	7.23	+10.20
10	0.0	7 600	1.0216	53 758	0.5102	1147.6	12.36	+9.86

* For Points 0, reactions = 0 except $V = 1$.

The reactions due to temperature change are computed, as follows:

$$H_t \frac{EI_c}{t\,\alpha} = \frac{l}{\Delta_{HL}\,EI_c} + \frac{H_V\,l}{\Delta_{VL}\,EI_c} = 0.018916 \text{ (Equation (30))}$$

$$V_t \frac{EI_c}{t\,\alpha} = \frac{h}{\Delta_{VL}\,EI_c} + \frac{V_H\,l}{\Delta_{HL}\,EI_c} = 0.001425 \text{ (Equation (31))}$$

$$M_t \frac{EI_c}{t\,\alpha} = H_t \frac{EI_c}{t\,\alpha}(r_1 - e) - V_t \frac{EI_c}{t\,\alpha}(l_1 - c) = 0.4995 \text{ (Equation (32))}$$

In similar manner the reactions due to rib-shortening may be computed with Equations (33) to (37), inclusive, taking values of C'_m and C''_m from Fig. 12.

DISCUSSION OF UNSYMMETRICAL ARCH DESIGN

The method given in this paper establishes the influence lines for moments and thrusts in an unsymmetrical arch when the crown or thinnest section is at the top of the arch where the tangent to the rib axis is horizontal. The thickness need not be the same at the two springings. If one springing is raised much above the other, the unsymmetrical arch will probably be more satisfactory with the thinnest section at the top than at the center of the span. For both appearance and strength it will then ordinarily be better to make the rib thicker at the lower springing than at the upper.

If the high end is too near the top (that is, if $\frac{h}{r_1}$ is too great), it may be necessary to move the thinnest section nearer the center of the span, in order to avoid excessive moments at the crown and at the high end or an excessive rate of increase in rib thickness from the crown to the high end. Although the variation of the special conditions which control different arches is too great to be able to generalize accurately, the computations for maximum stresses in ribs of various proportions (omitted herein for the sake of brevity) indicate that even when $\frac{h}{r_1}$ is as high as $\frac{3}{4}$, the crown may be satisfactorily located at the top. Although, in this case, the live load bending is greater at the crown than at the center, the temperature, rib-shortening, and shrinkage moments are less at the crown. The actual relation between the maximum moments at the crown and center points in any particular case depends on the relative importance of the live load moments and the moments induced by temperature change, etc.

Assuming that the proper crown thickness has been established, a suitable thickness for the rib at the springing lines can be computed ordinarily by placing m equal to about 0.3 in the formula, $m = \frac{d^3_c}{d^3_s \cos \phi_s}$.

Since the cos ϕ_s is greater at the high end than at the low end, using the same value of m for both ends will result in a greater value of d_s at the low end than at the high end. In an arch similar to that described in the "Numerical Example," but with $\frac{h}{r_1} = \frac{3}{4}$, this method gave values of d_s equal to 40 in. and 37 in., and the calculated maximum stresses were found to be very nearly equal at the two ends. Many designers have a tendency to make the springings too thick in relation to the crown thickness.

The relations between a symmetrical arch and certain type of unsymmetrical arch, shown by Carl B. Andrews,[15] Assoc. M. Am. Soc. C. E., are valuable in making a preliminary design for an unsymmetrical arch and for the final design of an arch which is so nearly symmetrical that the assumed form is satisfactory.

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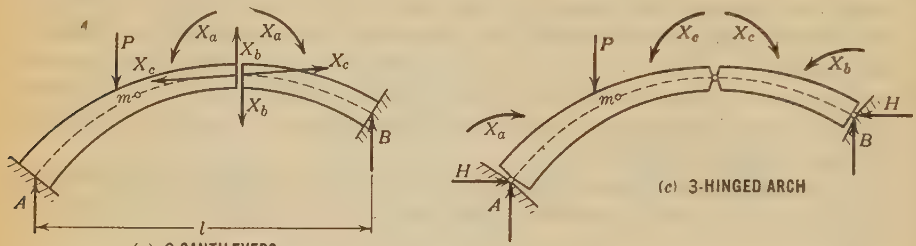
[15] "Reactions for a Particular Type of Unsymmetrical Arch," by Carl B. Andrews, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 438.

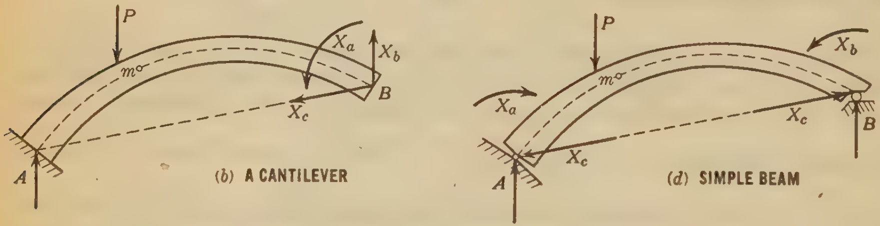
DISCUSSION

David A. Molitor,[16] M. Am. Soc. C. E. (by letter).—In this paper as in the previous one on design of symmetrical concrete arches,[3] the author uses the method of elastic weights for determining the influence lines for the reactions, and the problem of the unsymmetrical arch is solved by treating two halves of two different symmetrical arches.



(a) 2 CANTILEVERS

(c) 3-HINGED ARCH



(b) A CANTILEVER

(d) SIMPLE BEAM

Fig. 14.—Fixed Arches. Four Different Principal Systems, Each with Three Redundants

Fixed Arches.—The problem of fixed arches involves three redundant conditions and there are at least four possible different assumptions as to choice of a principal system. These four cases are illustrated in Fig. 14, for which the redundant conditions are, as follows:

Fig. 14(*a*).—Moment, X_a; shear, X_b; and thrust, X_c.
Fig. 14(*b*).—Moment, X_a; shear, X_b; and thrust, X_c.
Fig. 14(*c*).—Moment, X_a; moment, X_b; and moment, X_c.
Fig. 14(*d*).—Moment, X_a; moment, X_b; and thrust, X_c.

The redundant conditions, X_a, X_b, X_c, represent forces or moments, respectively, as shown, and are externally applied to the principal system in each case. The three redundants are additional to the reactions produced by the actual dead and live loads, P, and all together, the redundants, reactions, and loads, must form a system in equilibrium, when applied to any one of the four principal systems as shown.

Neglecting the effect of axial thrust, the three redundants may be evaluated from three elasticity equations, expressing the relations between the external loads, P, and the redundants, X, as follows:

$$\frac{X_a}{E\,I}\int M^2_a\,du + \frac{X_b}{E\,I}\int M_a M_b\,du + \frac{X_c}{E\,I}\int M_a M_c\,du = \frac{1}{E\,I}\int M_{om} M_a\,du\,. \quad (42a)$$

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[16] U. S. Constr. Engr., for Superv. Archt., U. S. Treasury Dept., Lansing, Mich.

$$\frac{X_a}{E\,I}\int M_b M_a\,du + \frac{X_b}{E\,I}\int M^2{}_b\,du + \frac{X_c}{E\,I}\int M_b M_c\,du = \frac{1}{E\,I}\int M_{om} M_b\,du \;..(42b)$$

and,

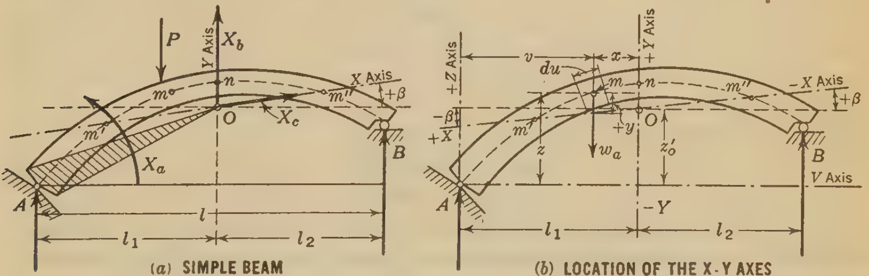
$$\frac{X_a}{E\,I}\int M_c M_a\,du + \frac{X_b}{E\,I}\int M_c M_b\,du + \frac{X_c}{E\,I}\int M^2{}_c\,du = \frac{1}{E\,I}\int M_{om} M_c\,du \;..(42c)$$

The general equation for moment about any axial point, *m,* for any case of simultaneous loads, *P,* is expressed by:

$$M_m = M_{om} - M_a\,X_a - M_b\,X_b - M_c\,X_c \ldots\ldots\ldots\ldots(43)$$

in which, X_a X_b, and X_c, are evaluated from Equations (42), and M_{om} is the moment of the external forces, *P,* about the axial point, *m,* of the principal system, when X_a, X_b, and X_c are not acting. Also, M_a, M_b, and M_c are moments about the same axial point, *m,* for the respective conventional loads, $X_a = 1$, $X_b = 1$, and $X_c = 1$, acting separately on the principal system, when the loads, *P,* are absent. Hence, all quantities in Equations (42) and (43), except the three *X*'s, are statically determinate by any of the four methods illustrated, and the three elasticity Equations (42), finally determine these redundants, X_a, X_b, X_c.

The author attempts the general solution of fixed unsymmetrical arches on the assumptions in Fig. 14(*a*), and immediately encounters difficulties because of the interruption in his principal system at the crown of the arch. These difficulties are entirely avoided when the principal system in Fig. 14(*d*) is chosen, since all equations then apply over the entire span, instead of suffering the discontinuity between the two cantilevers as in Fig. 14(*a*).



(*a*) SIMPLE BEAM (*b*) LOCATION OF THE X-Y AXES

FIG. 15.

Simplifications.—The simplifications possible in dealing with the principal system (Fig. 14(*d*)), are illustrated in Fig. 15. Referring to Fig. 15(*a*), the redundant conditions are moment, X_a; shear, X_b; and thrust, X_c, applied to the rigid disk, *AO.* The point, *O,* is the center of gravity of all the elastic weights, $w_a = \dfrac{du}{E\,I}$. The redundants (Fig. 15(*a*)) are shifted to a point, *O,* such that only those terms in Equations (42), involving M_{om}, $M^2{}_a$, $M^2{}_b$, and $M^2{}_c$ remain. All other terms become zero, thus avoiding the necessity of solving the three elasticity Equations (42), for simultaneous values of the X's. By introducing the elastic weights, $w_a = \frac{du}{EI}$, $w_b = x w_a$, and $w_c = y w_a$, into the remaining terms of Equations (42) and noting that $M_a = 1$, $M_b = x$, and $M_c = y \cos \beta$, as defined previously herein, the following simple relations are obtained after some reduction:

$$X_a = \frac{\Sigma M_{om} w_a}{\Sigma w_a} \quad \text{.......................} (44a)$$

$$X_b = \frac{\Sigma M_{om} w_b}{\Sigma x w_b} \quad \text{.......................} (44b)$$

and,

$$X_c = \frac{\Sigma M_{om} w_c}{\cos \beta \, \Sigma y w_c} \quad \text{.......................} (44c)$$

in which the integrations have been replaced by summations.

Fig. 15(a) shows the new disposition of the redundants when chosen so as to simplify the problem and avoid the solution of the three elasticity equations, while yet retaining the simple beam, $A B$, for a principal system.

The redundants are now applied to a rigid disk, $A O$, attached to the left end of the beam. A similar set of redundants, X, of equal and opposite kind (but not shown), is assumed acting at the right end of the beam.

The point, O, which is the center of gravity of the elastic weights, w_a, is located by taking moments of all the weights about two assumed axes, as Z, V, with origin, at A, to find the co-ordinates, l_1 and z'_0, as follows:

$$l_1 = \frac{\Sigma v w_a}{\Sigma w_a} \quad \text{.........................} (45)$$

$$z'_0 = \frac{\Sigma z w_a}{\Sigma w_a} \quad \text{.........................} (46)$$

and,

$$\tan \beta = - \frac{\Sigma x z w_a}{\Sigma x^2 w_a} \quad \text{......................} (47)$$

The new axes of the co-ordinates, with the origin, O, may now be drawn with the Y-axis vertical and the X-axis making an angle, β, with the horizontal, such that the centrifugal moment, $\Sigma x y w_a$, of the elastic weights, w_a, about O must be zero. (See Fig. 15(b).) For symmetrical arches, $l_1 = l_2 = \frac{1}{2} l$, and $\beta = O$; and, the ordinate, $z'_0 = \frac{\Sigma z w_a}{\Sigma w_a}$, alone locates the co-ordinate axes, X, Y, with the origin at O.

With the redundants chosen so that X_b is the vertical component along the Y-axis, X_c, the thrust along the X-axis, and X_a, a moment applied to the disk, $A O$, Equation (43) may be reduced to its simplest terms.

Referring to Fig. 15(b), the following values are apparent from the foregoing definitions: $M_a = 1$ is the moment about any axial point, m, produced by the moment, $X_a = 1$, applied to the principal system; $M_b = 1\ x$ is the moment about this same point, m, due to the force, $X_b = 1$, acting on the principal system; and, $M_c = 1\ y \cos \beta$ is the moment about this same point, m, due to the force, $X_c = 1$, acting on the principal system. These values introduced into Equation (43) give the general moment equation for any axial point, m, of an unsymmetrical arch, as:

$$M_m = M_{om} - 1\ X_a - x_m\ X_b - y_m \cos \beta\ X_c \ldots\ldots\ldots\ldots (48)$$

in which, M_{om} is the moment about the axial point, m, due to the actual loads, P, acting on the principal system. The point, m, has the co-ordinates, x_m, y_m. When the arch is symmetrical the angle, β, becomes zero and $\cos \beta = 1$.

The foregoing outline thus affords a complete solution of the geometry of all fixed arch problems by one and the same set of equations, and following the same method of analysis, except that the unsymmetrical case involves more labor.

Critical Stresses.—For the unsymmetrical arch critical stresses will occur at the five sections, A, m', n, m'', and at B, as may be shown by discussing the values for M_m as given by Equation (48) for these five sections.

For a symmetrical arch only three sections, A, m', and n, involve critical stresses. The critical stresses represent the maxima of all possible maximum stresses, thus rendering unnecessary the stress analysis for any arch sections other than those herein designated.

Importance of Shape of Axial Line as Affecting Economic Design.—The most important single factor in economic arch design is the shape of the arch axis, which should be a three-centered or five-centered curve as nearly as possible coincident with the line of thrust drawn for total dead load plus one-half the equivalent uniform live load over the entire span. The dead-load pressure line advocated by the author, and approximated by a transformed catenary, will not result in equal positive and negative moments for the critical sections, and is, therefore, not the shape for maximum economy.

Concluding Remarks.—The writer has aimed to emphasize the importance of the fundamental assumptions which lead to the most acceptable solution of the general problem of fixed arches whether symmetrical or otherwise.

The error of the author in starting out with a solution for symmetrical arches which is well enough as far as it goes, certainly leads to unnecessary complications when adapted to the unsymmetrical form. It is far better to solve the geometry in the simplest possible manner for the unsymmetrical case and then introduce the slight simplifications occasioned by the special case of symmetry, as the writer has done in this discussion. In other words, the solution herein indicated is applicable to any arch problem, the only difference being in the extra numerical work which is always involved in solving the unsymmetrical case. Further details of the stress analysis are not pertinent to a discussion of this paper. The writer has presented details of preliminary design and final analysis in another connection.[17]

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[17] "Kinetic Theory of Engineering Structures," by David A. Molitor, 1911, Chapter XV, on "Fixed Masonry Arches."

The author claims: (*a*) That the simplicity of his method of analysis permits a more thorough study of the proportions of an unsymmetrical arch; (*b*) that this should result in a more satisfactory design; and (*c*) that an attempt is made to clarify the action of the unsymmetrical arch so that each step can be readily understood by the designer. The author has apparently failed to substantiate these claims.

CHARLES S. WHITNEY,[18] M. AM. SOC. C. E. (by letter).—With the exception of certain unsupported statements which the writer wishes to question, there appears to be nothing in Mr. Molitor's discussion to indicate that the writer's method for the analysis of unsymmetrical arches is not much more simple in application than the orthodox method presented by Mr. Molitor. The latter requires that the rib dimensions be determined completely and that the values of elastic weights and their functions be calculated before the reactions and stresses are determined. The writer's method permits the calculation of reactions and stresses without the necessity of determining first the rib thickness and axis co-ordinates at all points. For this reason, it is possible, by the writer's method, to calculate the stresses in a number of ribs and to arrive at the most economical design in less time than is required for the complete analysis by the orthodox method of one assumed design which, in itself, will not indicate the most economical proportions.

The writer did not begin with a solution for symmetrical arches (as stated by Mr. Molitor), but he did "solve the geometry in the simplest possible manner for the unsymmetrical case." The writer's solution of the symmetrical arch is based on the elastic system used by Mr. Molitor with the rib cut at the abutment only; but for the unsymmetrical case, because of the discontinuity of the arch-axis equation at the crown, it is simpler to cut the rib at the crown, treating the two sides of the rib separately. No difficulties were encountered because of the interruption of the principal system at the crown of the arch.

The writer does not entirely agree with Mr. Molitor that the shape of the arch "should be a three-centered or five-centered curve as nearly as possible coincident with the line of thrust drawn for total dead load plus one-half the equivalent uniform live load over the entire span." As a matter of fact, that method does not have the effect of equalizing positive and negative live load moments. However, if it were desired to use the pressure line for dead load plus a part of the live load, it may be done simply by including the proper live load in the values, w_c and w_s. This is discussed by the writer in his previous paper.[19]

It is not possible, by changing the shape of the arch, to equalize positive and negative moments at all critical sections. Mr. Molitor's statement as to the location of critical stresses at certain definite sections appears somewhat broad, as their location may be affected by load concentrations and the relative importance of stresses due to rib-shortening, temperature changes, etc.

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[18] Cons. Engr., Milwaukee, Wis.

[19] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 966.

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INTERCEPTING SEWERS AND STORM STAND-BY TANKS AT COLUMBUS, OHIO

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WITH DISCUSSION BY MESSRS. C. B. HOOVER AND C. D. MCGUIRE, JULIAN MONTGOMERY, D. T. MITCHELL, ROBERT SPURR WESTON, W. W. HORNER, SAMUEL A. GREELEY, AND JOHN H. GREGORY, ORRIS BONNEY, AND ROBERT A. ALLTON.

SYNOPSIS

The sewerage system of a city is designed, in general, to meet certain assumed conditions with the thought that when these conditions have been reached, a new plan will be developed to provide additional facilities.

In 1908, the City of Columbus, Ohio, completed comprehensive sewerage improvements designed to give adequate facilities for some years to come. The time arrived when the capacity of these works was reached or exceeded, and it became necessary to provide additional facilities not only to give adequate sewerage to the city but also to meet higher sanitary standards. A program was adopted, therefore, designed to give adequate facilities for estimated 1960 conditions. This paper describes that part of the program which has been completed and is now (1933) in operation.

The works described consist of (*a*) intercepting sewers with sufficient capacity so that overflows of storm sewage to the streams are limited to a predetermined number per year; (*b*) regulator chambers to control the quantity of storm sewage entering these sewers; and (*c*) storm stand-by tanks to give partial treatment to all storm sewage in the intercepting sewers over and above the quantity that can be handled by sewage treatment works.

NOTE.—Published in October, 1933, *Proceedings*.

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INTRODUCTION

In 1908 the City of Columbus completed sewerage improvements[5] consisting of certain trunk sewers, sewage and storm-water pumping stations, force mains, and sewage treatment works. Since 1908 the city has enjoyed a steady growth in population and area; in 1930, the population, according to the United States Census, was 290 564 and the area, 38.8 sq miles.

With increasing population and sewage flow, the need for additional facilities was forcibly brought to the attention of the City Officials by the unsightly and polluted condition of the Olentangy and Scioto Rivers and of Alum Creek, both within and below the city (see Fig. 1). These conditions were officially brought to the attention of the Ohio Department of Health by officials and residents of counties located down stream, with the result that formal action was taken by the State of Ohio in 1927 whereby the City was ordered "* * * to install such works or means as may be necessary for collecting and disposing of the sewage of the City in a manner to correct and prevent the pollution of the Scioto River and Alum Creek * * *." The Olentangy River is tributary to the Scioto near the center of the city.

The general plan adopted in compliance with this order involves the construction of intercepting and sanitary sewers, regulator chambers on combined sewers, the conversion of certain districts sewered on the combined system to districts sewered on the separate system, the construction of storm stand-by tanks on the Scioto River and on Alum Creek, and the construction of additional sewage treatment facilities.

As a part of the general plan, the City has constructed, since 1927, about eight miles of intercepting sewers, eight regulator chambers for intercepting predetermined volumes of flow from combined sewers, storm stand-by tanks at Whittier Street, on the Scioto River, and a similar tank at East Main Street, on Alum Creek, all of which have been placed in service, and it is the purpose of this paper to describe the design and construction of these works.

SEWER DISTRICTS AND COLLECTION OF SEWAGE

The city, and adjacent territory in Franklin County, is divided naturally into the three main sewer districts shown in Fig. 1, the East Side District, the Intercepting Sewer District, and the West Side District.

East Side District.—Originally, the East Side District was provided with sewers on the combined system (sanitary and storm), but in recent years extensions have been made on the separate system. The principal outlets for the district are the 9-ft Main Street combined sewer, which had a storm overflow discharging into Alum Creek, and the 42-in. sanitary sewer, which discharges into one of the new intercepting sewers. Prior to the construction of the works described herein the dry-weather flow in the Main Street Sewer was diverted to the East Side Sewage Pumping Station and from there pumped over a divide into the Intercepting Sewer District.

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[5] "The Improved Water and Sewage Works of Columbus, Ohio," by John H. Gregory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), p. 206.

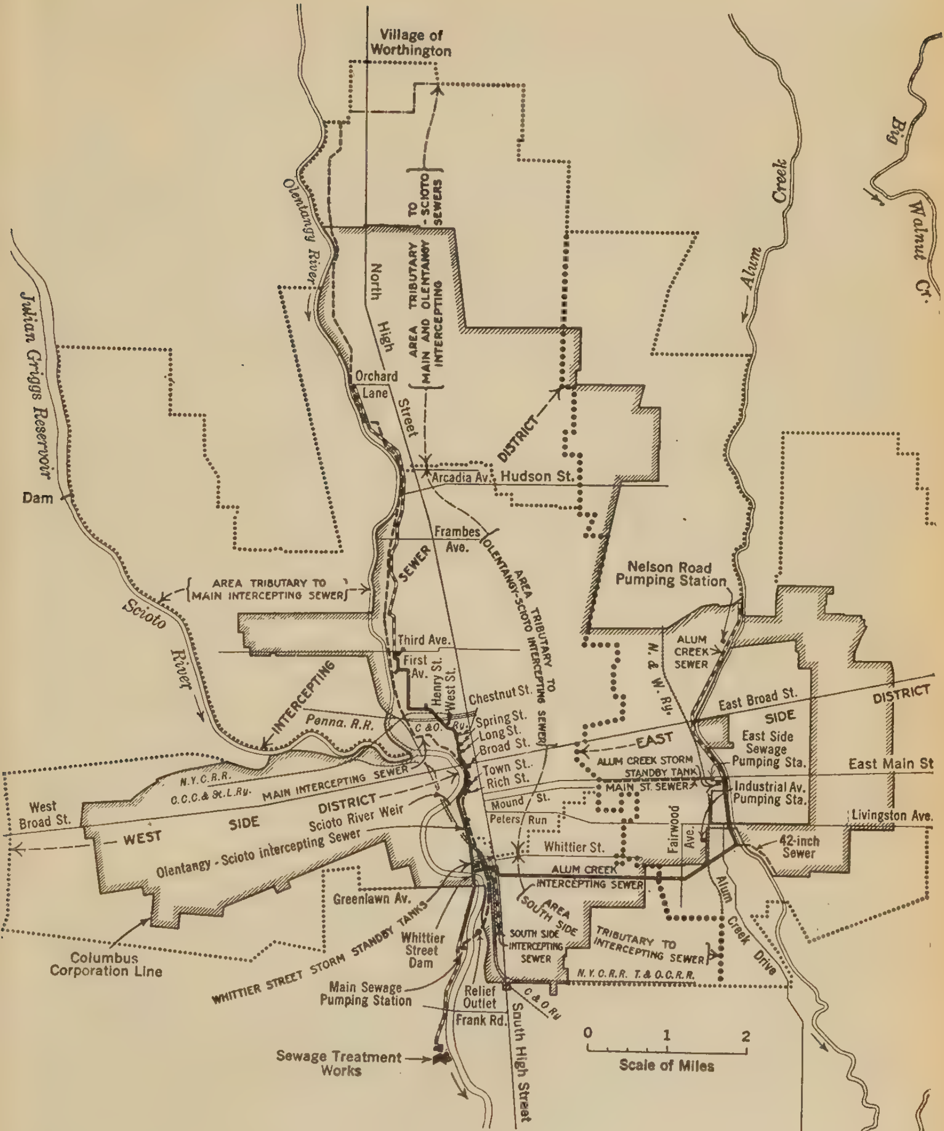
Village of Worthington
TO - SCIOTO SEWERS
AREA TRIBUTARY TO MAIN AND OLENTANGY - SCIOTO INTERCEPTING
Olentangy River
North High Street
Orchard Lane
Big Walnut Cr.
Alum Creek
Julian Griggs Reservoir
Dam
DISTRICT
Arcadia Av.
Hudson St.
Frambes Ave.
SEWER
AREA TRIBUTARY TO OLENTANGY-SCIOTO INTERCEPTING SEWER
AREA TRIBUTARY TO MAIN INTERCEPTING SEWER
Nelson Road Pumping Station
Scioto River
Third Ave.
First Av.
Henry St.
West St.
INTERCEPTING
ALUM CREEK SEWER
N. & W. Ry.
Chestnut St.
Penna. R.R.
C & O. Ry.
Spring St.
Long St.
Broad St.
East Broad St.
DISTRICT
SIDE
EAST
East Side Sewage Pumping Sta.
ALUM CREEK STORM STANDBY TANK
East Main St
N.Y.C.R.R.
C.C.C. & St.L.Ry.
MAIN INTERCEPTING SEWER
Town St.
Rich St.
MAIN ST. SEWER
Industrial Av. Pumping Sta.
West Broad St.
DISTRICT
SIDE
WEST
Scioto River Weir
Mound St.
Peters Run
Livingston Ave.
Olentangy - Scioto intercepting Sewer
Whittier St.
Fairwood Ave.
42-inch Sewer
ALUM CREEK INTERCEPTING SEWER
Greenlawn Av.
SOUTH SIDE AREA TRIBUTARY TO INTERCEPTING SEWER
SOUTH SIDE INTERCEPTING SEWER
Alum Creek Drive
Columbus Corporation Line
WHITTIER STREET STORM STANDBY TANKS
Whittier Street Dam
N.Y.C.R.R. T. & O.C.R.R.
Main Sewage Pumping Station
Relief Outlet
C & O Ry
Frank Rd.
South High Street
0 1 2
Scale of Miles
Sewage Treatment Works

FIG. 1.—MAP OF COLUMBUS, OHIO, AND VICINITY.

There were two pumping stations—on Nelson Road and Industrial Avenue —from which small quantities of sewage were pumped into sewers tributary to the Main Street Sewer. The East Side Sewage and Industrial Avenue Pumping Stations were shut down so that the sewage would flow by gravity from the East Side District through a new sewer to the Intercepting Sewer District. The Nelson Road Pumping Station will remain in service pending the construction of a sewer to connect the small area now served by this station with one of the new intercepting sewers.

Intercepting Sewer District.—Sewerage in the Intercepting Sewer District is partly on the combined and partly on the separate system. West of the Olentangy River the District is on the separate system, as is also that part east of the river and, in general, north of Arcadia Avenue (see Fig. 1); the remainder of the District is on the combined system of sewerage. Prior to the completion of the new works the sanitary sewage, together with the dry-weather flow from the combined sewers, was discharged into the Main Intercepting Sewer through which, with the dry-weather flow pumped from the East Side District, it flowed to the Main Sewage Pumping Station.

West Side District.—The West Side District is largely on the separate system, the dry-weather flow being conveyed to the Main Sewage Pumping Station through two trunk sewers[5] known as the West Side Sanitary Sewer and the Old 4-Ft Dry Flow Sewer. Storm water, for the most part is discharged into open watercourses tributary to the Scioto River.

County Sewer Districts.—The inclusion in the three main Sewer Districts of territory located outside the corporate limits of the city (Fig. 1), has come about partly by reason of the establishment, under the Sanitary Sewer District Law of Ohio, of county sewer districts[6] whereby the sewage from these districts is discharged, by agreement, into the sewerage system of the City.

Use of Separate and Combined Sewers.—Since 1923, it has been the policy of the City to build new sewers on the separate system, wherever possible.

EXISTING SEWERAGE WORKS

The earliest sewers of record were constructed in 1853. Of these, two are of particular interest—the Main Street Sewer and the Main Intercepting Sewer (Fig. 1).

Main Street Sewer.—In early days the Main Street Sewer discharged into Alum Creek, but beginning in 1908 the dry-weather flow was diverted and pumped to the Intercepting Sewer District. Ordinarily, during dry weather, it was the practice to pump the entire sewage flow. During storms, however, it was possible to pump only a small part of the storm flow and, as a result, storm sewage has been discharged, untreated, into Alum Creek. With the completion of the works described herein, storm sewage overflows to Alum Creek only after receiving partial treatment by sedimentation.

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[6] "County Sewer District Work in Ohio and Assessment of Cost According to Benefits," by E. G. Bradbury, M. Am. Soc. C. E., *Transactions* Am. Soc. C. E., Vol. 94 (1930), p. 445.

Main Intercepting Sewer.—The Main Intercepting Sewer varies from 15 to 72 in. in diameter. With the sewer flowing full, and with $n = 0.015$ in Kutter's formula, velocities of less than 2.5 ft per sec obtain over approximately one-half the length of the sewer, the minimum being 1.8 ft per sec in sections 1 500 and 750 ft long, respectively. As a result, and with sewage from combined sewers, deposits have occurred which have had to be removed at intervals. In the future this line will be a sanitary sewer, and one important reason for converting it was the low velocities obtaining; furthermore, the sewer has been found to be of inadequate capacity at various points. Most of the connections between the Main Intercepting Sewer and the combined sewers were small pipes varying from 6 to 15 in. in diameter and extending from sumps, 1 ft or 2 ft deep, in the bottom of the combined sewers, to the intercepting sewer. In general, there were no means of closing these connections, or of controlling the volume of flow through them. At three connections, sand-catchers had been constructed in order to decrease the quantity of sand and grit entering the intercepting sewer.

Main Sewage Pumping Station.—Normally, in dry weather, all the sewage flowed to the Main Sewage Pumping Station from which it was pumped to the sewage treatment works. During periods of high water in the Scioto River, however, sewage was pumped either to the treatment works, or to the river, and, at times, it overflowed to the river through a relief outlet on the Main Intercepting Sewer, near the pumping station. During 1932 the average volume pumped was 36 000 000 gal daily. All the sewage, however, was not pumped, and it is estimated that actually the sewage flow was about 37 000 000 gal daily.

Sewage Treatment Works.—The sewage treatment works, shown in Fig. 1, are of the Imhoff tank-trickling filter type. For a number of years the works have been greatly overloaded, with the result that the treatment afforded has been insufficient to produce a satisfactory effluent.

Flow of Rivers and Streams

By reference to Fig. 1, it will be seen that two rivers, the Olentangy and the Scioto, and a smaller stream, Alum Creek, pass through the city.[7] In the summer and early fall months, the flow in these streams is usually small, often, however, attaining flood proportions in the late fall, winter, and spring months. During the summer and early fall months, practically all the flow of the Scioto is taken for water-supply purposes, with the result that the flow through the central part of the city during such periods consists of but little more than the run-off of the Olentangy.

As illustrative of stream flows, it may be stated that, in 1930, there were periods of 2, 4, 10, 20, and 31 consecutive days when the flow in the Scioto through the central part of the city, where the water-shed is about 1 610 sq miles, was less than 4, 12, 20, 36, and 48 cu ft per sec, respectively. During periods of such low flows the corresponding velocities of flow at West Broad Street, in the pool above the Scioto River Weir, do not exceed 0.02 ft per sec. Ordinarily, the flow in the Olentangy, with a water-shed of 536 sq miles, would be somewhat less than that of the Scioto through the central part of the city. Similarly, on Alum Creek, in 1930, there were 3, 11, 16, 27, and 62 consecutive days when the flow at Livingston Avenue, where the water-shed is 190 sq miles, was less than 4, 5, 10, 14, and 19 cu ft per sec, respectively, with corresponding velocities of flow in the stream varying from 0.2 to 0.5 ft per sec.

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[7] For a map of Columbus and adjacent water-sheds, see "The O'Shaughnessy Dam and Reservoir," by John H. Gregory, C. B. Hoover, and C. B. Cornell, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1432.

These values emphasize the fact, that, with such low flows and velocities, only a little sewage, even if diluted with storm water, could be discharged into the streams, in warm weather, as they flow through Columbus, without creating objectionable conditions.

Overflow of Sewage to Rivers and Streams

From the combined sewers in the Intercepting Sewer District there were nineteen storm overflow outlets to the Olentangy and Scioto Rivers. These combined sewers flow in general in westerly directions toward the two rivers and range in size, at the overflow outlets, from 24 in. to 10 ft 6 in. in diameter.

In times of storm, the volume of flow passing through the intercepting connections to the Main Intercepting Sewer was limited by the capacity of the intercepting sewer and by the size of the connections. As a result, storm sewage overflowed to the rivers and it has been estimated that there were about thirty-five such overflows per year from each of the combined sewers. Most of the intercepting connections, built many years ago, were of a type easily clogged, and, following storms, many of them were found to be completely closed, at frequent intervals, thus permitting sewage to overflow until the connections were cleaned.

The Main Intercepting Sewer was barely of sufficient capacity to handle the average rate of dry-weather sewage flow. With a maximum rate of flow during certain hours of the day, and at certain points along the sewer, sewage, unable to enter the sewer already flowing full, and at times under pressure, overflowed to the rivers. Furthermore, as already stated, sewage entered the Scioto River, at times, through the Relief Outlet.

In the East Side Sewer District, in addition to the outlet near East Main Street, there were three outlets north of this point from which overflows occurred, but during storms only.

Pollution of Rivers and Streams

As stated previously, in a number of places the sewage or storm sewage could overflow to the streams. As a result, unsightly conditions and offensive odors were caused by the stranding, sedimentation, and subsequent putrefaction of heavy solids and floating materials which enter the streams during such periods of overflow. In addition, offensive conditions in the Scioto River below the sewage treatment works were brought about by the lack of capacity of the works to treat, satisfactorily, all the dry-weather sewage flow.

As a result of an investigation of stream conditions in and near Columbus, the Division of Sanitary Engineering of the Ohio Department of Health submitted a report on this subject, in 1927, to the officials of the State Health Department. The following brief summaries, based on statements made in this report, will serve to give a general idea of stream conditions as observed by the Division at the time of its investigation.

Olentangy River.—Above the city, both the Olentangy and the Scioto Rivers were in a normal condition and were not polluted. Below the northerly limits of the city, however, the effect of storm overflows from combined sewers was quite marked, particularly in the lowering of the dissolved oxygen content of the river water.

Scioto River.—Through the central part of the city, and as far south as Frank Road, storm overflows from the combined sewers, and from the Relief Outlet, made the Scioto River unsightly, at certain points, due to discoloration of the water and to stranded solids and oil accumulations along the banks. At other points, decomposing sludge on the bed of the river gave rise to gas ebullition and to floating masses of sludge. At Frank Road the dissolved oxygen content was low, the stream was discolored, and odors could be detected. Below Frank Road the effluent from the treatment works is discharged into the river, and it was stated that the effluent was black and odorous, and that it contained suspended matter in some quantity. The effect of the discharge of the effluent was evident at points below the works, gross pollution, with a practical absence of dissolved oxygen, extending to points 5 and 7 miles south. From Big Walnut Creek, 8 miles below the treatment works, to Big Darby Creek, 21 miles below the works, the river gradually returned to a normal condition.

Alum Creek.—Above East Main Street, Alum Creek presented a clean appearance and analytical determinations disclosed no sewage pollution. Commencing at East Main Street, however, gross pollution occurred, which was attributed to sewage entering the creek from the storm overflow at the end of the Main Street Sewer. At times, during dry weather, the dissolved oxygen content approached zero at East Main Street and also at a point $\frac{3}{8}$ mile down stream. At a point about 2 miles below East Main Street the creek began to improve in appearance, and a normal dissolved oxygen content was reached at its confluence with Big Walnut Creek, about 4 miles farther south.

MEANS ADOPTED FOR RELIEVING INSANITARY CONDITIONS IN THE SCIOTO RIVER AND IN ALUM CREEK

The problem of correcting and preventing the pollution of the streams resolved itself into two parts, one having to do with conditions within and immediately adjacent to the city (attributed to inadequate sewer capacity), thereby resulting in sewage overflows, storm overflows, and clogged intercepting connections, and the other, with conditions below the city, resulting mainly from insufficient treatment facilities.

In the design of works for the correction and prevention of the pollution within and immediately adjacent to the city it was decided: (1) That new sewers of adequate capacity to prevent the overflow of sewage should be provided; (2) that the greatest possible reduction, consistent with reasonable cost, should be made in the number of storm overflows; and (3) that a type of intercepting connection be adopted that would be as free from clogging as possible.

In the case of the Olentangy and Scioto Rivers, compliance with the foregoing requirements necessitated the design and construction of two intercepting sewers, known as the Olentangy-Scioto Intercepting Sewer and the South Side Intercepting Sewer. They were to be of capacities sufficient to handle the dry-weather flow of sewage for a reasonable period in the future, and, in addition, to have capacities for storm sewage flows considerably greater than have heretofore ordinarily been provided in intercepting sewers. Having made provision in these intercepting sewers for relatively large volumes of storm sewage, the problem of disposing of this sewage presented itself. With a maximum rate of flow of approximately 487 000 000 gal daily, there was no question but that the storm sewage should be directed to some point down stream from the central part of the city. It was obviously not feasible to attempt to handle all the flow at the treatment works; and to discharge into the river, down stream, the flow in excess of that to be handled by the treatment works would only remove to some point below the central part of the city the condition that was being corrected up stream. It was decided, therefore, that, near Whittier Street, some means of treatment should be provided for the excess flow of storm sewage prior to its discharge into the Scioto River.

On Alum Creek the problem was concentrated at East Main Street to a large extent, and it was decided that here, too, the excess flow of storm sewage from the Main Street Sewer should be given some treatment before being discharged into the creek.

Year by year the sewage flow from the East Side District increased and, in 1929, following the annexation to the city of a large area east of Alum Creek, plans were made for directing sewage from the City of Bexley and a large area in Franklin County into the City System. Consequently, it was decided that the time had come to cease pumping sewage from the East Side District and to build a gravity sewer south along Alum Creek and thence west to the Olentangy-Scioto Intercepting Sewer. As a result of this decision, the new sewer, known as the Alum Creek Intercepting Sewer, was built. Sewage flows through it and is discharged into the Olentangy-Scioto Intercepting Sewer just below the storm stand-by tanks near Whittier Street. (See Fig. 1.)

The new works comprising that part of the general plan for correcting and preventing the pollution of the streams within and immediately adjacent to the city consist of:

(1) The conversion of the Main Intercepting Sewer to a sanitary sewer, to receive the flow from existing and future sanitary sewers in that District.

(2) The Olentangy-Scioto Intercepting Sewer—to intercept sewage and large volumes of storm sewage from combined sewers in the Intercepting

Sewer District north of Whittier Street. This will reduce the number of overflows to the Olentangy and Scioto Rivers, and will convey the flow through the central part of the city to storm stand-by tanks at Whittier Street, from which point a controlled volume will flow to the sewage treatment works.

(3) The South Side Intercepting Sewer—to intercept sewage and large volumes of storm sewage in the Intercepting Sewer District south of Whittier Street, thereby reducing the number of overflows to the Scioto River. This will convey the flow to the Olentangy-Scotio Intercepting Sewer just above the storm stand-by tanks at Whittier Street.

(4) Regulator chambers—to control the flow of storm sewage from combined sewers to the Olentangy-Scioto and South Side Intercepting Sewers.

(5) The Whittier Street Storm Stand-By Tanks—to give partial treatment by sedimentation to storm sewage flows in the Olentangy-Scioto Intercepting Sewer in excess of controlled volumes of flow to the sewage treatment works, before discharge into the Scioto River.

(6) The Alum Creek Intercepting Sewer—to intercept sewage from the East Side District and to convey it to the Olentangy-Scioto Intercepting Sewer just below the Whittier Street tanks.

(7) The Alum Creek Sewer—to collect sanitary sewage from areas in the East Side District north of East Main Street (now on the separate system) and to convey it to the Alum Creek Intercepting Sewer just below the storm stand-by tank at East Main Street.

(8) The conversion of certain areas in the Intercepting Sewer District, and in the East Side District north of East Broad Street (now on the combined system), to the separate system, by the construction of sanitary sewers.

(9) The Alum Creek Storm Stand-By Tank—to give partial treatment by sedimentation to storm sewage flows in the Main Street Sewer in excess of controlled volumes of flow into the Alum Creek Intercepting Sewer, before discharge into Alum Creek.

General Bases of Design of New Sewerage Works

Period of Design.—It was assumed that the intercepting sewers and regulator chambers should be of capacity sufficient to serve until 1960. The Whittier Street tanks were designed for estimated conditions as of 1945, with provision for extension to meet conditions as of 1960. Because of the smaller dry-weather flow in the Main Street Sewer, and because the area tributary is largely developed at the present time, the storm stand-by tank at Alum Creek was designed for estimated conditions as of 1960. However, provision was made for another tank should it prove desirable later.

Area Assumed to Contribute Sewage in 1960.—To relieve insanitary conditions in the streams involves the handling of sewage from areas immediately adjacent to the city as well as from the city proper. In assuming the area to contribute sewage in 1960 (Fig. 1), consideration was given to known factors, recognizing, however, that possible changes in industrial and economic conditions, as well as other factors, may effect the boundaries of the area. The total area assumed was 86.6 sq miles, subdivided as follows:

	Square miles
Area tributary to both the Main and Olentangy-Scioto Intercepting Sewers	14.2
Area tributary to Main Intercepting Sewer	12.1
Area tributary to Olentangy-Scioto Intercepting Sewer	10.9
East Side District	33.0
Area tributary to South Side Intercepting Sewer	4.9
West Side District	11.5
Total	86.6

Population.—A house-to-house canvass of a number of typical blocks in various sections of the city was made and data were obtained as to the density of population. With these data as a guide, and with the U. S. Census records, it was estimated that the population on the area assumed to contribute sewage in 1945 will be 484 000, and that, in 1960, it will be 643 000. The distribution of population in 1945, and in 1960, is shown in Table 1(*a*). In distributing the 1960 population, densities varying from 5 persons per acre in suburban areas, to 33 persons per acre in the most built-up sections of the city, were used.

TABLE 1.—BASIC ESTIMATES BY SEWER DISTRICTS, IN 1945 AND 1960

Year (1)	East Side District: Alum Creek Intercepting Sewer (2)	INTERCEPTING SEWER DISTRICT				West Side District (7)	Total, Columns (2), (6), and (7) (8)
		Main Intercepting Sewer (3)	Olentangy-Scioto Intercepting Sewer (4)	South Side Intercepting Sewer (5)	Total, Columns (3), (4), and (5) (6)		
			(*a*) POPULATION				
1945	108 500	65 200	179 970	46 730	291 900	83 600	484 000
1960	157 000	108 600	209 800	55 600	374 000	112 000	643 000
		(*b*) AVERAGE DRY-WEATHER SEWAGE FLOW, IN MILLION GALLONS DAILY					
1945	10.9	6.4	27.8	6.4	40.6	8.1	59.6
1960	15.7	10.9	34.0	8.2	53.1	11.2	80.0

Sewage Flow.—After studying the records of sewage pumped at the Main Sewage Pumping Station, it was assumed that the average sewage flow from the area contributing sewage in 1960 would be 125 gal per capita per day, or a total of about 80 000 000 gal daily. In residential districts on the separate system of sewerage a flow of 100 gal per capita daily was assumed, whereas, in districts on the combined system, within which are located most of the commercial and industrial developments, the flow was assumed at 185 gal per capita daily.

In the design of the sewers, maximum rates of sewage flow were assumed as never less than twice the average rate, and for small districts as great as four times the average rate of flow, as shown on a curve[8] developed for the

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[8] "Two Diagrams for Use in Designing Sanitary Sewers," by James R. McComas, *Engineering News-Record,* Vol. 96, (June 10, 1926), p. 951.

Metropolitan Sewerage Commission of New York City in 1910 by one of the writers. The estimated average dry-weather sewage flows in 1945 and in 1960 are shown in Table 1(*b*).

Storm-Water Flow.—In the design of intercepting sewers it has been common practice to make provision for some storm-water to care for the first flushings of streets and sewers. Such provision has usually been made by adding an arbitrary quantity to the assumed rate of sewage flow per capita.

In the design of the Olentangy-Scioto and South Side Intercepting Sewers, however, one of the governing conditions was that the greatest possible reduction, consistent with reasonable cost, be made in the number of storm overflows from the combined sewers tributary to these intercepting sewers, thus making it necessary to provide for considerably greater volumes of storm-water than have usually been handled in intercepting sewers.

Inasmuch as the design of these two sewers was largely a problem of storm-water design, the so-called rational method for estimating storm-water flow was used. In the application of this method the following values of the run-off coefficient, *c*, were, in general, assumed: 0.75 for commercial areas; 0.45 for industrial areas; 0.35 for residential areas; and 0.15 for parks and railroad yards. The inlet time was taken at 10 min.

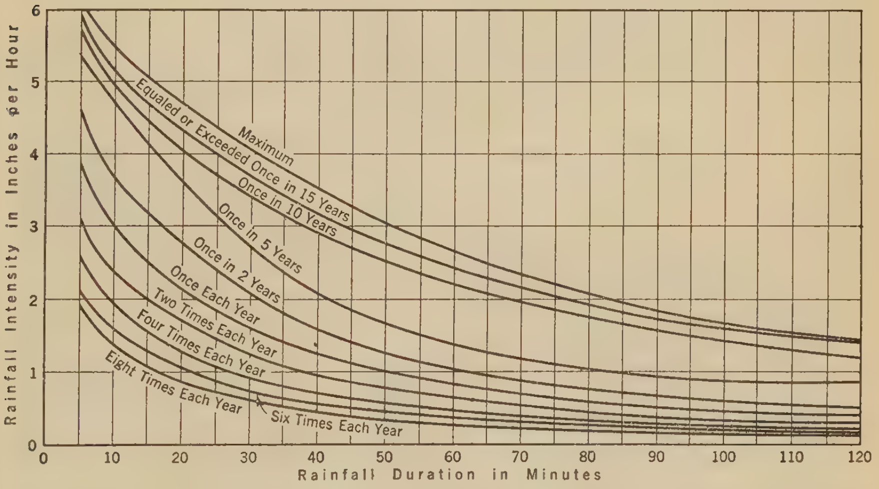


FIG. 2.—INTENSITY, DURATION, AND FREQUENCY RAINFALL CURVES FOR COLUMBUS, OHIO.

Rainfall intensities in Columbus were computed, for different durations and frequencies, from records of the United States Weather Bureau for the years, 1897 to 1923, inclusive. The curves of Fig. 2, were developed from a study of the rainfall intensity.

Studies and estimates of cost, using different frequencies of rainfall, were made and, as a result, in the computation of the volumes of storm-water to be handled by the Olentangy-Scioto and the South Side Intercepting Sewers, the rainfall intensity curves used were such that, on the average, the combined sewers in the central part of the city, from the Olentangy River to West Mound Street, will not overflow more than four times per year to the Scioto River, and along the Olentangy River and along the Scioto below West Mound Street, not more than eight times per year.

Hydraulics of Sewers.—In determining the sizes of the new intercepting sewers (concrete with vitrified clay lining), Kutter's formula, $n = 0.013$, was used, whereas, in computing the capacities of existing sewers, n was taken as 0.015. All sewers were assumed to flow full. As the flow in the intercepting sewers is largely from combined sewers, slopes were adopted which, in general, give velocities of not less than 2.5 ft per sec for present average rates of dry-weather flow. In one stretch of the Olentangy-Scioto Intercepting Sewer a velocity of approximately 2.25 ft per sec was used, and, in another, where the sewer carries sanitary sewage only, a velocity of approximately 2 ft per sec was assumed. For estimated 1960 conditions, however, these velocities will be not less than 2.5 ft per sec.

THE OLENTANGY-SCIOTO INTERCEPTING SEWER

The Olentangy-Scioto Intercepting Sewer, as planned, extends from the Sewage Treatment Works to Orchard Lane (see Fig. 1). A profile is shown in Fig. 3. The estimated rates of sewage flow in this sewer in 1945 and in 1960 are given in Table 2(*a*).

TABLE 2.—ESTIMATED RATES OF SEWAGE FLOW IN 1945 AND 1960.

Item No.	Location	RATES OF FLOW, IN MILLION GALLONS DAILY									
		Average								Maximum assumed for design	
		1945				1960				1945	1960
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
	(*a*) IN THE OLENTANGY-SCIOTO INTERCEPTING SEWER										
1	Olentangy-Scioto Intercepting Sewer above the South Side Intercepting Sewer connection	27.8	....	....	....	34.0	....	....	....	......	......
2	South Side Intercepting Sewer	6.4	....	....	....	8.2	....	....	....	......	......
3	At Whittier Street Storm Stand-by Tanks (Item No. 1 plus Item No. 2)	....	34.2	....	....	....	42.2	....	....	68.4	84.4
4	Main Intercepting Sewer	....	6.4	....	....	....	10.9	....	....	......	......
5	Alum Creek Intercepting Sewer	....	10.9	....	....	....	15.7	....	....	......	......
6	Whittier Street Storm Stand-by Tanks to Main Sewage Pumping Station (sum of Items Nos. 3, 4, and 5)	....	....	51.5	....	....	....	68.8	....	103	138
7	West Side Sanitary and Old 4-Foot Dry Flow Sewers	....	....	8.1	....	....	....	11.2	....	......	......
8	Main Sewage Pumping Station to Sewage Treatment Works (Item No. 6 plus Item No. 7)	....	....	....	59.6	....	....	....	80.0	120	160
	(*b*) IN THE ALUM CREEK INTERCEPTING SEWER										
9	Alum Creek Sewer above Main Street	2.8	....	....	....	4.0	....	....	....	......	......
10	Main Street Sewer	5.0	....	....	....	6.7	....	....	....	......	......
11	At Alum Creek Storm Stand-by Tank (Item No. 9 plus Item No. 10)	....	7.8	....	....	....	10.7	....	....	15.0	22.3
12	Area east of Alum Creek *	....	2.5	....	....	....	4.0	....	....	......	......
13	At Livingston Avenue (Item No. 11 plus Item No. 12)	....	....	10.3	....	....	....	14.7	....	21.7	29.8
14	Industrial Avenue Sewer and area south-east of the City and west of Alum Creek	....	....	0.6	....	....	....	1.0	....	......	......
15	At Whittier Street Storm Stand-by Tanks (Item No. 13 plus Item No. 14)	....	....	....	10.9	....	....	....	15.7	22.6	31.5

* Includes Village of Hanford, west of Alum Creek.

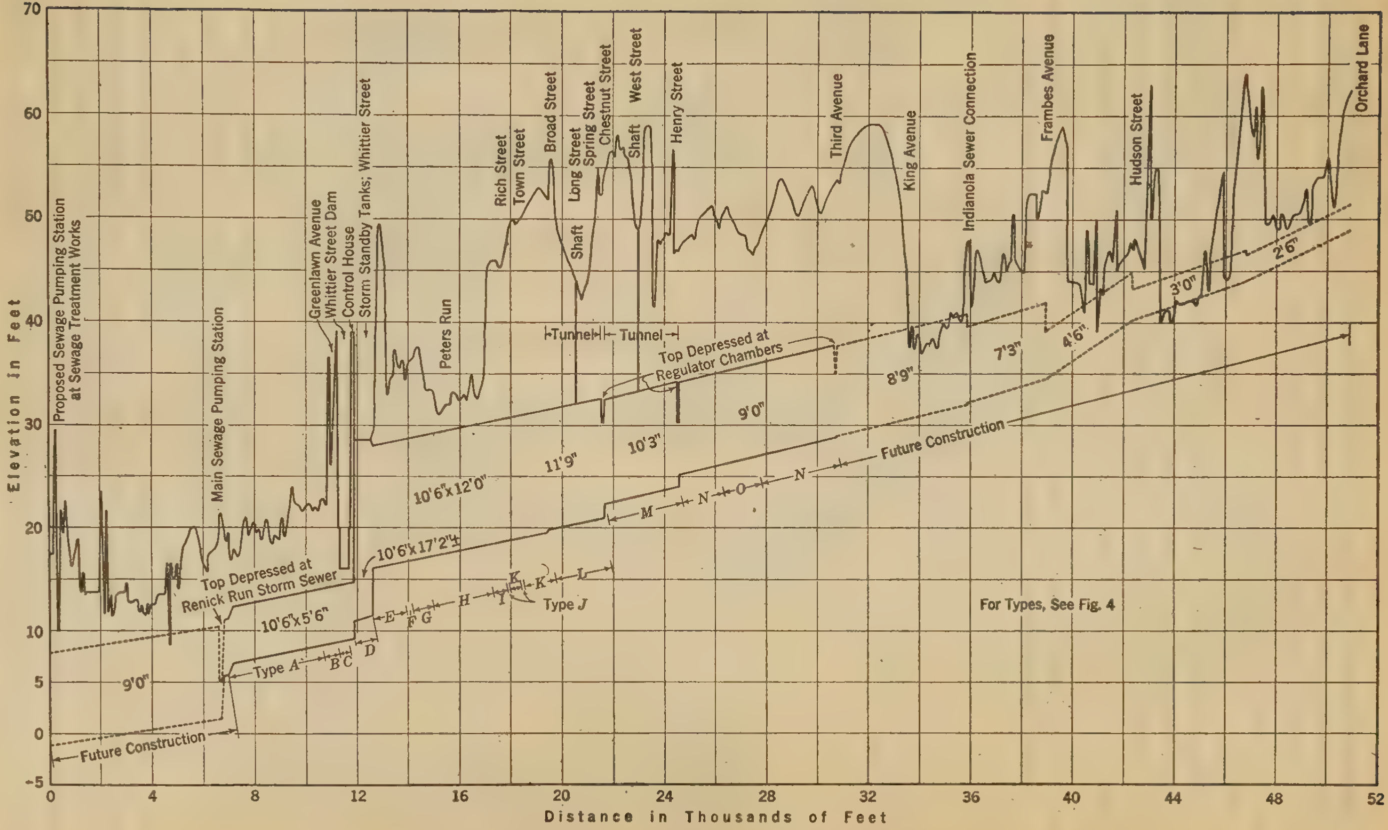


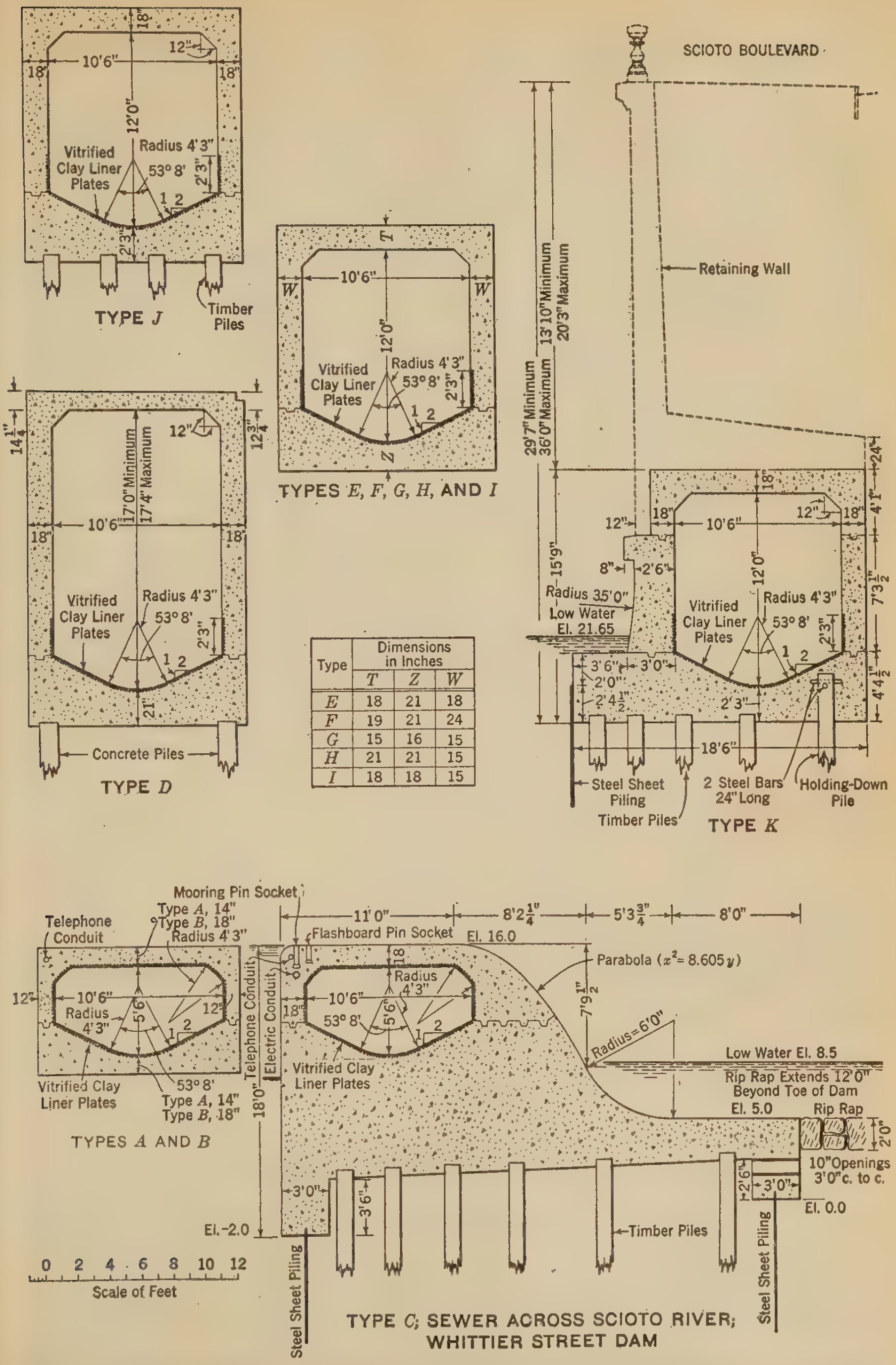
FIG. 3.—PROFILE OF OLENTANGY-SCIOTO INTERCEPTING SEWER, COLUMBUS, OHIO.

Storm Sewage Flow.—Storm sewage, from the combined sewers now connected, is discharged into this intercepting sewer at the regulator chambers. At the Whittier Street tanks a part of the flow is passed on to the Main Sewage Pumping Station, the remainder being diverted into the storm stand-by tanks for sedimentation.

Capacity has been provided for maximum rates of dry-weather sewage flow from all the areas assumed tributary to this sewer in 1960 plus controlled maximum rates of storm flow from the combined sewers in the Intercepting Sewer District, the maximum rate of storm-water flow at each collection point having been added to the maximum rate of dry-weather sewage flow to obtain the total maximum rate of flow for design. The estimated maximum rates of flow under 1960 conditions and the corresponding total capacities, in gallons per capita daily, are shown in Table 3. Storm sewage from the combined sewers is discharged into this intercepting sewer beginning at Hudson Street, and from this point to the Whittier Street tanks. In this

TABLE 3.—Estimated Rates of Storm Sewage Flow in the Olentangy-Scioto Intercepting Sewer in 1960 and Corresponding Capacity, in Gallons per Capita per Day.

Collection point	Rates of Flow Beginning at Collection Points, in Million Gallons Daily				Estimated population tributary in 1960	Total capacity provided, in gallons per capita per day
	Sewage		Maximum storm-water flow	Total flow (figures rounded)		
(1)	Average dry-weather flow (2)	Maximum dry-weather flow (3)	(4)	(5)	(6)	(7)
Orchard Lane	3.5	9.1	...	9	35 000	257
Brighton Road	3.7	9.4	...	9	37 000	243
Weber Road	4.0	9.8	...	10	40 000	250
Sunset Drive	5.1	12.2	...	12	51 000	235
Glen Echo	5.4	12.6	...	13	54 000	241
Hudson Street	5.9	13.5	20	34	56 500	600
Frambes Avenue	8.2	18.0	97	115	69 400	1 660
Woodruff Avenue	8.4	18.0	97	115	70 000	1 640
Indianola Sewer Connection	11.9	24.5	162	187	89 000	2 100
King Avenue	12.5	25.5	162	188	92 100	2 040
Fourth Avenue	12.6	26.0	162	188	92 800	2 030
Third Avenue	15.8	31.8	173	205	110 000	1 870
Second Avenue	16.0	32.0	173	205	111 000	1 850
Henry Street	21.0	42.0	226	268	138 000	1 940
West Street	23.3	46.6	238	285	151 000	1 890
Chestnut Street	27.7	55.4	289	344	174 000	1 980
Spring Street	27.9	55.8	310	366	175 000	2 090
Long Street	28.2	56.4	314	370	177 000	2 090
Broad Street	29.3	58.6	328	387	183 000	2 110
Capital Street	29.3	58.6	328	387	183 000	2 110
State Street	29.3	58.6	328	387	183 000	2 110
Town Street	29.4	58.8	328	387	183 000	2 110
Rich Street	29.6	59.2	328	387	185 000	2 090
Peters Run Sewer Connection	34.0	68.0	339	407	208 000	1 960
South Side Intercepting Sewer Connection	42.2	84.4	403	487	265 000	1 840
Whittier Street Storm Stand-by Tanks:						
Main Intercepting Sewer Connection	53.1	106	...	106	374 000	284
Alum Creek Intercepting Sewer Connection	68.8	138	...	138	531 000	259
West Side Sanitary Sewer Connection	80.0	160	...	160	643 000	249


18"
12"
10'6"
18"
18"
12'0"
Vitrified Clay Liner Plates
Radius 4'3"
53° 8'
2'3"
1 2
2'3"
TYPE J
Timber Piles
T
10'6"
W
W
12'0"
Vitrified Clay Liner Plates
Radius 4'3"
53° 8'
2'3"
1 2
Z
TYPES E, F, G, H, AND I
SCIOTO BOULEVARD
Retaining Wall
29'7" Minimum
36'0" Maximum
13'10" Minimum
20'3" Maximum
15'9"
Radius 35'0"
Low Water El. 21.65
12"
8"
2'6"
18"
18"
10'6"
12"
18"
24"
4'1"
12'0"
7'3½"
Vitrified Clay Liner Plates
Radius 4'3"
53° 8'
2'3"
1 2
4'4½"
3'6"
3'0"
2'0"
2'4½"
2'3"
18'6"
Steel Sheet Piling
Timber Piles
2 Steel Bars 24" Long
Holding-Down Pile
TYPE K
14¼"
12¾"
12"
17'0" Minimum
17'4" Maximum
10'6"
18"
18"
Vitrified Clay Liner Plates
Radius 4'3"
53° 8'
2'3"
1 2
21"
Concrete Piles
TYPE D
Mooring Pin Socket
Type A, 14"
Type B, 18"
Radius 4'3"
Telephone Conduit
12"
10'6"
Radius 4'3"
5'6"
12"
1 2
Vitrified Clay Liner Plates
53° 8'
Type A, 14"
Type B, 18"
TYPES A AND B
11'0"
8'2¼"
5'3¾"
8'0"
Flashboard Pin Socket
El. 16.0
18
Radius 4'3"
10'6"
18"
53° 8'
5'6"
1 2
Telephone Conduit
Electric Conduit
Parabola ($x^2 = 8.605y$)
7'9½"
Radius=6'0"
Low Water El. 8.5
Rip Rap Extends 12'0" Beyond Toe of Dam
El. 5.0
Rip Rap
2'0"
10" Openings 3'0" c. to c.
Vitrified Clay Liner Plates
18'0"
3'0"
3'6"
Timber Piles
2'6"
3'0"
El. 0.0
El. -2.0
Steel Sheet Piling
Steel Sheet Piling
0 2 4 6 8 10 12
Scale of Feet
TYPE C; SEWER ACROSS SCIOTO RIVER; WHITTIER STREET DAM

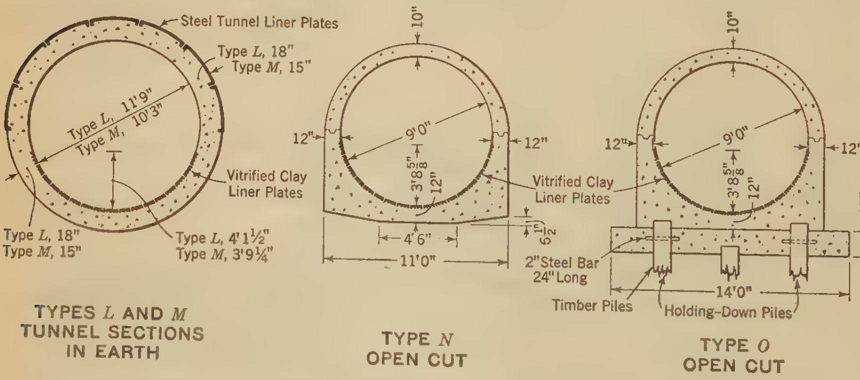
Type	Dimensions in Inches		
	T	*Z*	*W*
E	18	21	18
F	19	21	24
G	15	16	15
H	21	21	15
I	18	18	15

FIG. 4.—OLENTANGY-SCIOTO INTERCEPTING SEWER, COLUMBUS, OHIO, RECTANGULAR SEWER SECTIONS.

section the capacity varies from 600 to 2 110 gal, with an average of about 1 900 gal, per capita daily, a greater capacity than has heretofore been provided in intercepting sewers, as far as the writers are aware.

Distance Sewage Travels and Corresponding Time.—The farthest any sewage will travel to reach the Whittier Street tanks is from Worthington, a distance of 10.1 miles. For the first 2.9 miles sewage will flow through the Main Intercepting Sewer. For the remaining 7.2 miles, and with the completion of the Olentangy-Scioto Intercepting Sewer to Orchard Lane, the flow will be divided between these two sewers. For sewage to reach the storm stand-by tanks from Worthington, 6.8 hr under present conditions, and 5 hr under 1960 conditions, will be required for average rates of dry-weather sewage flow. To reach the treatment works sewage will flow an additional 2.6 miles, making a total distance from Worthington of 12.7 miles. To reach the treatment works from Worthington a period of 8.1 hr will be required under present conditions, and 6.1 hr in 1960, for average rates of dry-weather flow.

Sewer Sections.—In selecting the cross-sections of the sewer, studies were made of rectangular sections of both the single and twin-barrel type with **V**-shaped inverts, arch sections with **V**-shaped inverts, circular sections, and sections with a cunette. Velocities under present minimum rates of flow required that special attention be given to the shape of the invert, especially south of the central part of the city. As a result, single-barrel rectangular sections with **V**-shaped inverts were used from the Main Sewage Pumping Station to West Broad Street. North of West Broad Street to Third Avenue, where the sewer was specified to be built in tunnel, either outright or as an alternate, circular sections were adopted as being best suited to meet the conditions anticipated. North of West Third Avenue, and from the pumping station to the treatment works, circular sections are planned. Sections of the rectangular sewers, Types *A* to *K*, inclusive, are shown on Fig. 4, and sections of the circular sewers, Types *L* to *O*, inclusive, on Fig. 5. The retaining wall on Type *K* (Fig. 4) was built subsequent to



TYPES *L* AND *M* TUNNEL SECTIONS IN EARTH

TYPE *N* OPEN CUT

TYPE *O* OPEN CUT

FIG. 5.—OLENTANGY-SCIOTO INTERCEPTING SEWER, CIRCULAR SEWER SECTIONS.

the construction of the sewer, which was designed structurally and architecturally to support the wall.

Types A and B.—Although the interior dimensions of Types *A* and *B* are the same, it was necessary to vary the exterior dimensions and the amount of steel reinforcment to meet different conditions of loading. Both types were designed to resist an internal pressure of about 33 ft of water above the invert, a condition which might occur should the pumping station be shut down temporarily. Back-filling was not permitted until the concrete in the top slab was at least 21 days old.

Type C.—Whittier Street Dam.—At the Scioto River the Type *C* sewer (Fig. 4), was extended across the river within a low overflow dam, approximately 520 ft. long, to avoid the use of an inverted siphon. At the sub-grade of the dam the river bed is pervious gravel. Experience at Columbus, with bridges and other structures founded on such material, has indicated the necessity of pile foundations and precautions to prevent under-cutting during flood-stages. The structure, therefore, was supported on piles and was protected against under-cutting by cut-off walls of steel sheet-piling. At both ends of the dam, the lines of sheet-piling were connected, thus forming a complete cut-off.

The structure is stable, when the sewer is either empty or full: (*a*) With the pool above the dam empty; (*b*) with the pool full to the top of the flash-boards at Elevation 20.0; (*c*) with the pool full to the top of the flash-boards and with upward pressure; and (*d*) with the pool overflowing, with the water surface at Elevation 38.0. Under each of these conditions the resultant force strikes the base within the middle-third. The most severe combination of conditions is with the sewer empty, with the pool full to the top of the flash-boards, and with upward pressure, when the factor of safety against overturning is 5.2. Upward pressure was assumed as full-head pressure at the heel, decreasing uniformly to zero at the toe, and acting on two-thirds of the area of the base.

The dam was constructed in 30-ft sections within an earth coffer-dam. Driving of the sheet-piling cut-offs was continued until, with the equipment and method used, further penetration was not possible. The penetrations obtained, however, were such as to indicate that the piling extended into blue clay, shown by borings to underlie the gravel. The bearing value of the piles was computed by the *Engineering News* formula, and they were driven until a safe bearing value of at least 20 tons per pile was obtained.

Provision has been made at the north end for by-passing a part of the flow of the river, or for lowering the water level in the pool, through four 4-ft square passages passing under the intercepting sewer. Double rows of stop-planks have been provided at both ends of the by-pass structure which is founded on 14-in. square pre-cast reinforced concrete piles.

Type D.—This type (Fig. 4) was constructed along the west side of the storm stand-by tanks near Whittier Street. With the sewer empty, with the levee immediately west of it in place, and with no support on the east side (a condition possible not only during the construction of the present installation, but also when additional tanks are built), the structure would not be in equilibrium against side-thrust. Therefore, two rows of pre-cast reinforced concrete piles were driven under the sewer to which the structure was anchored. These piles also serve to hold the structure down when hydrostatic uplift is developed with the sewer empty and with high water in the river. Leading to each storm stand-by tank are four inlet openings, 4 ft wide by 4 ft 2 in. high, the bottoms of which are approximately 7 ft 3 in. above the invert of the sewer. There is, also, an outlet opening, with its bottom about 2 ft 3 in. above the invert of the sewer, through which material deposited in the tank is flushed into the sewer. Openings, with bulkheads, have also been provided for two additional tanks.

Types E, F, G, H, I, and J.—Each of these types (Fig 4) was designed not only to support external loads but also to resist an internal pressure of about 17 ft of water above the invert, which would obtain with the gates of the storm stand-by tanks shut off and with the regulator and emergency outlet gates in the Control House closed. Back-filling was not permitted until the concrete in the top slab was 21 days old.

Type K.—Type *K* (Fig. 4) was designed to support a retaining wall, with back-filling, and is supported on piles. To resist overturning, the structure was anchored to each pile in the row under the heel. Protection against erosion was provided by driving a steel sheet-pile cut-off wall along the river face of the concrete base. The structure was constructed within an earth coffer-dam. The retaining wall shown on Type *K* was built subsequent to the construction of the sewer, the sewer having been designed structurally and architecturally to carry the retaining wall.

Types L and M.—Under the railroad tracks north of Chestnut Street, Type *M* was built of reinforced concrete; otherwise, Types *L* and *M* (Fig. 5) were constructed of plain concrete. Each type was built through glacial drift, in tunnel, under compressed air. The air pressures used varied from 3 to 8 lb per sq in., and voids behind the tunnel lining were filled by pressure grouting. On Type *L*, the grout averaged 0.18 cu yd, and on Type *M*, 0.20 cu yd, per ft of sewer.

Types N and O.—These types (Fig. 5) were built in open trenches, Type *N* where the sub-soil provided sufficient bearing power, and Type *O* where the bearing power was insufficient.

SOUTH SIDE INTERCEPTING SEWER

The South Side Intercepting Sewer will connect with the Olentangy-Scioto Intercepting Sewer at West Whittier Street, just above the storm stand-by tanks (Fig. 1). It varies in diameter from 63 to 72 in. and is for the purpose of conveying sewage from one district served by separate sewers and from two districts served by combined sewers. Capacity is provided for maximum rates of dry-weather sewage flow from the area assumed to be tributary in 1960 plus controlled maximum rates of storm-sewage flow from the two combined sewer districts, the maximum rate of storm-water flow at each collection point having been added to the maximum rate of dry-weather sewage flow to obtain the total maximum rate of flow for design.

REGULATOR CHAMBERS

The dry-weather sewage flow from each combined sewer connected to one of the new intercepting sewers passes through a regulator chamber. During storms, however, the rate of flow of storm sewage is limited by the setting of a sluice-gate on the connection leading to the intercepting sewer. The storm sewage, in excess of that which passes into the intercepting sewer, accumulates and rises in the regulator chamber until it passes over the overflow weirs and thence through a channel leading directly to the river, or back to the combined sewer below the regulator chamber, which piece of sewer serves as a storm overflow. The sluice-gate serves not only to limit the flow to the intercepting sewer but also to shut off the flow should that be desired. With the connection closed, sewage, in dry weather, or storm sewage, during storms, can pass over the overflow weirs. Under normal operating conditions the sluice-gate is partly open and set to limit the flow to that desired. The regulator chambers were designed to limit the number of storm overflows to not more than 4 per year, on the average, in the central part of the city, where combined sewer outlets to the rivers are relatively close together, and not more than 8 per year, on the average, north and south of the city, where outlets are more widely scattered. Details of the Broad Street Regulator Chamber are shown in Fig. 6. Aside from the snow manhole, this regulator chamber is typical of those built at other locations.

Each regulator chamber is provided with electric lights set in water-tight marine fixtures and is also provided with hose connections for flushing. Stored in grooves below the floor are stop-planks so that the sluice-gate may be isolated for inspection and repairs. Two lengths of stop-planks were used, and a portable stop-plank lifter for each size has been provided.

To reduce the use of exposed iron work to a minimum, concrete balustrades were adopted instead of pipe railings. At the Chestnut Street Regulator Chamber an opening was provided through which a boat may be lowered into the intercepting sewer.

ALUM CREEK INTERCEPTING SEWER

The Alum Creek Intercepting Sewer (Fig. 1) was placed in service on March 15, 1932. A profile, and sections of the sewer, Types *P* to *S*, inclusive, are shown, respectively, on Figs. 7 and 8. Sewage from the East Side District flows in this sewer to and into the Olentangy-Scioto Intercepting Sewer just below the storm stand-by tanks near Whittier Street. During and following storms a controlled volume of storm sewage is discharged into the sewer at the storm stand-by tank on Alum Creek. The estimated rates of sewage flow in 1945 and in 1960 are given in Table 2(*b*). The capacity of the sewer, however, exceeds the 1960 requirements because it was constructed by tunneling, and construction requirements rather than capacity needs governed the size. Where not built by tunneling the sewer was made proportionally larger, to conform to the capacity provided in the tunnel. From East Livingston Avenue to the storm stand-by tanks it was constructed of concrete, and north of East Livingston Avenue, of vitrified clay segment block.

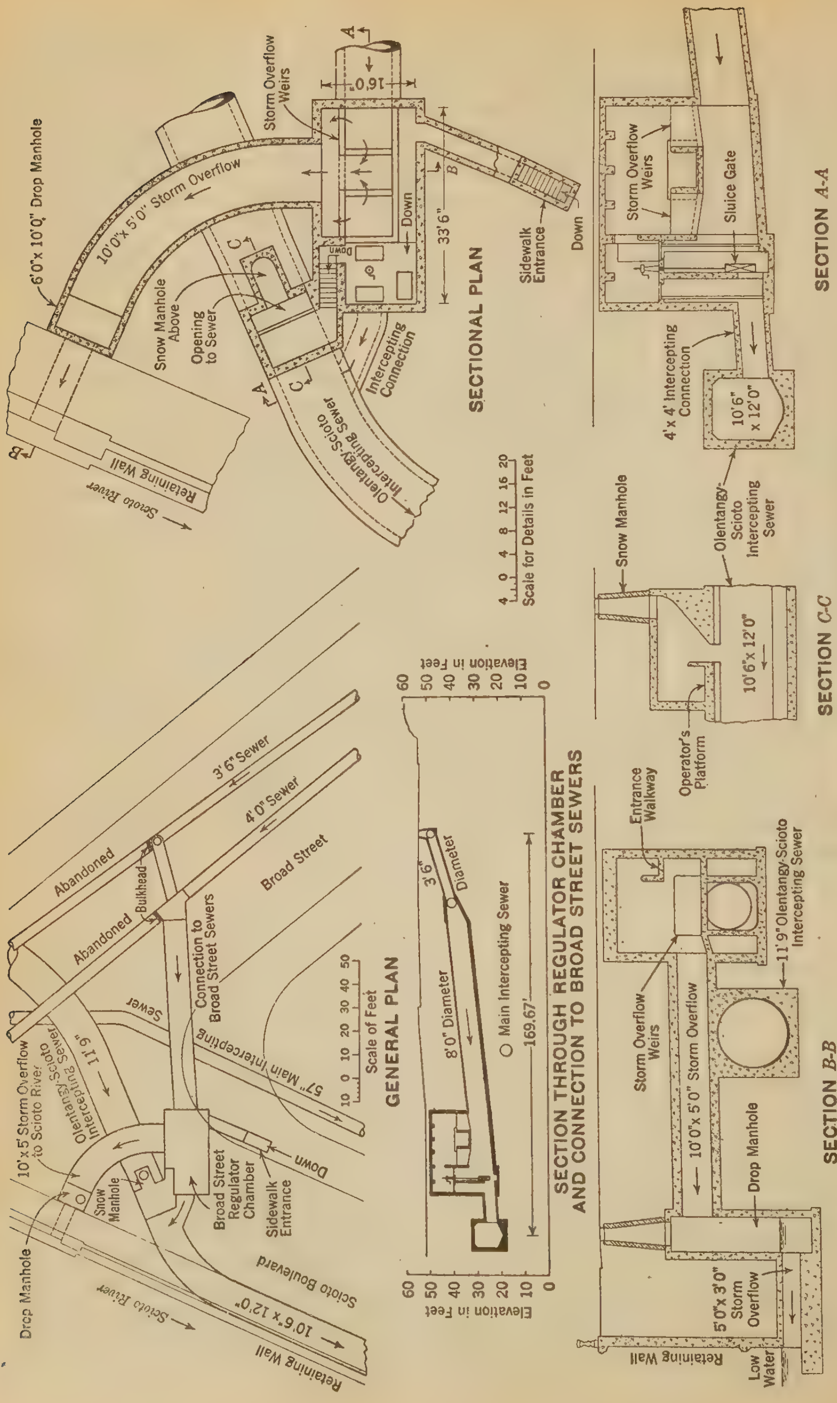

Drop Manhole
10'x5' Storm Overflow to Scioto River
Olentangy-Scioto Intercepting Sewer
11'9"
Snow Manhole
Broad Street Regulator Chamber
Sidewalk Entrance
Down
Scioto River
10'6" x 12'0"
Scioto Boulevard
Retaining Wall
Sewer
57" Main Intercepting
Connection to Broad Street Sewers
Bulkhead
Abandoned
Abandoned
3'6" Sewer
4'0" Sewer
Broad Street
Scale of Feet
10 0 10 20 30 40 50
GENERAL PLAN
Elevation in Feet
60 50 40 30 20 10 0
8'0" Diameter
3'6" Diameter
Main Intercepting Sewer
169.67'
SECTION THROUGH REGULATOR CHAMBER AND CONNECTION TO BROAD STREET SEWERS
6'0"x 10'0" Drop Manhole
10'0"x 5'0" Storm Overflow
Storm Overflow Weirs
16'0"
A
A
B
B
C
C
Snow Manhole Above
Opening to Sewer
Down
Down
33'6"
Sidewalk Entrance
Down
Intercepting Connection
Olentangy-Scioto Intercepting Sewer
Retaining Wall
Scioto River
SECTIONAL PLAN
4 0 4 8 12 16 20
Scale for Details in Feet
Storm Overflow Weirs
Sluice Gate
4'x 4' Intercepting Connection
10'6" x 12'0"
Olentangy-Scioto Intercepting Sewer
SECTION A-A
Snow Manhole
Operator's Platform
10'6"x 12'0"
SECTION C-C
Entrance Walkway
Storm Overflow Weirs
10'0"x 5'0" Storm Overflow
11'9" Olentangy-Scioto Intercepting Sewer
Drop Manhole
5'0"x 3'0" Storm Overflow
Low Water
Retaining Wall
SECTION B-B

FIG. 6.—DETAILS OF BROAD STREET REGULATOR CHAMBER, COLUMBUS, OHIO.

Types P and Q.—These types (Fig. 8) were built by tunneling under compressed air. Under the railroad, and in a few cases, where unusually heavy roof loads were encountered, Type *P* was built of reinforced concrete. Elsewhere, plain concrete was used. Most of the tunnels were constructed from seven shafts,[9] 1 700 to 2 400 ft apart. The headings varied from 644 to 1 380 ft in length, the average being 1 039 ft. Air pressures varied from 6 to 35 lb, the average being about 15 lb per sq in. Voids behind the tunnel lining were filled by pressure grouting, the grout averaging 0.14 cu yd per ft of sewer.

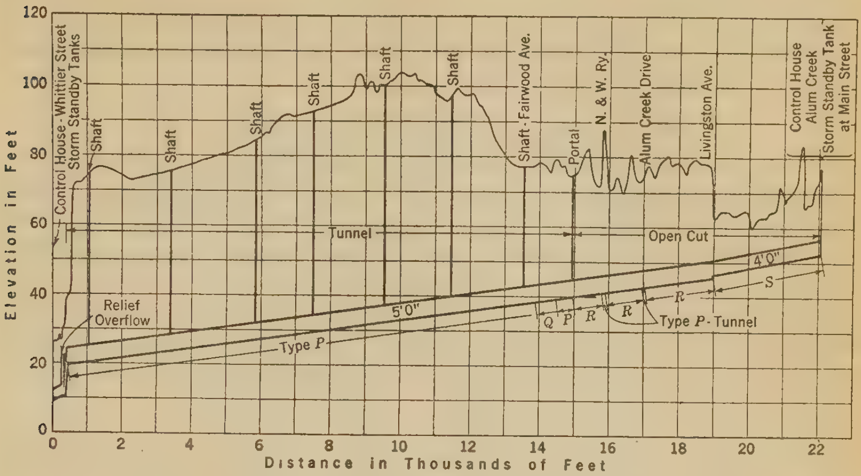


FIG. 7.—PROFILE OF ALUM CREEK INTERCEPTING SEWER, COLUMBUS, OHIO.

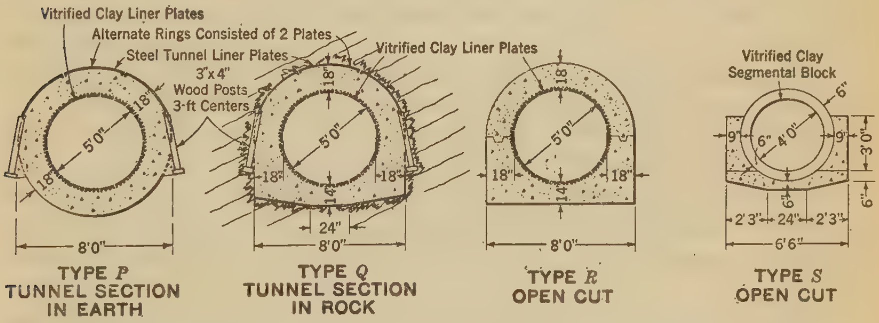

Vitrified Clay Liner Plates
Alternate Rings Consisted of 2 Plates
Steel Tunnel Liner Plates
3"x 4" Wood Posts 3-ft Centers
Vitrified Clay Liner Plates
18"
5'0"
18"
8'0"
TYPE P
TUNNEL SECTION IN EARTH
18"
5'0"
18"
18"
14"
24"
8'0"
TYPE Q
TUNNEL SECTION IN ROCK
18"
5'0"
18"
18"
14"
8'0"
TYPE R
OPEN CUT
Vitrified Clay Segmental Block
6"
9"
6"
4'0"
9"
3'0"
6"
6"
2'3"
24"
2'3"
6'6"
TYPE S
OPEN CUT

FIG. 8.—ALUM CREEK INTERCEPTING SEWER, CIRCULAR SEWER SECTIONS.

Types R and S.—Although the average depth of the sub-grade of Type *R* (Fig. 8) was approximately 35 ft, the sewer was built in open trench. Type *S* (Fig. 8) was also built in open trench.

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[9] "Surveying Under Compressed Air—Great Accuracy Obtained in Holing-Through Sewer Tunnel Headings at Columbus, Ohio," by Orris Bonney, M. Am. Soc. C. E., *Civil Engineering,* Vol. 3, (January, 1933), p. 31.

ALUM CREEK SEWER

The Alum Creek Sewer is shown in Fig. 1 extending from East Main Street to the north corporation line to collect sewage from two combined sewer districts which are to be converted to the separate system, and from areas to the north, now only sparsely populated. As planned, the sewer is to be 4 ft in diameter. The average rate of dry-weather flow for which provision has been made is 4 000 000 gal daily.

CONVERSION OF COMBINED SEWER DISTRICTS

In the Intercepting Sewer District a few small areas are served by combined sewers. Elevations of these sewers are such as not to permit of their connection to the Olentangy-Scioto Intercepting Sewer. In these areas the plans call for new sanitary sewers, tributary to the Main Intercepting Sewer, and the combined sewers are to be used for storm-water only. Similarly, in the East Side District, to eliminate two storm overflows north of East Broad Street, new sanitary sewers, tributary to the Alum Creek Sewer, are planned, and here, also, the combined sewers will be used for storm-water only.

FUNCTION OF STORM STAND-BY TANKS

The function of a storm stand-by tank is to remove heavy solids and floating materials from excess storm-sewage flows prior to their discharge into a body of water. Furthermore, with storms giving a volume of flow less than that needed to fill the tank and to cause over-flow, a storm stand-by tank acts as a detention basin, storing excess storm sewage at peak flows and returning it to the sewer after the storm is over. Ordinarily, the tank would be located adjacent to an intercepting sewer and at the point where excess storm flows would be diverted. Unless the tank is to be unwatered by pumping, its bottom should be above the sewage level in the intercepting sewer corresponding to the maximum rate of dry-weather flow, and in order that such a tank may function to the best advantage, means should be provided to control, automatically, the rate of flow to the sewer below the tank.

Requirements to Be Met.—The conditions that the storm stand-by tanks at Columbus were designed to fulfill are, as follows:

1.—To provide sedimentation of excess storm-sewage flows so as to remove from such sewage heavy suspended solids which, if allowed to enter the streams, might cause sludge banks and possible subsequent offensive conditions.

2.—To provide for the removal, from excess storm-sewage flows, of floating solids which might cause unsightly conditions if allowed to enter the streams.

3.—To drain the stored sewage back into the intercepting sewers after a storm is over and after the flow in the intercepting sewers has decreased to less than twice the average rate of dry-weather flow.

4.—To be capable of rapid and easy cleaning and to be arranged so that solids deposited in the tanks can be flushed back into the intercepting sewers and carried to the treatment works.

5.—To be capable of handling the entire storm flow in the intercepting sewers in case the treatment works should be temporarily out of service.

The conditions which the control works were designed to fulfill are:

1.—To provide automatic control of the volume of storm sewage flowing to the treatment works so that, at no time, will the capacity of the works be exceeded.

2.—At the Whittier Street tanks, to provide automatic control which will close off the tanks from the Scioto River during periods of extreme high water, at which time the tanks would otherwise be flooded.

Basis of Design.—As far as the writers are aware the storm stand-by tanks at Columbus were the first to be placed under construction in the United States. Owing to the absence of data on the operation of such tanks in this country, it was necessary to make certain assumptions. Those made, were

TABLE 4.—PERIODS OF DETENTION IN THE STORM STAND-BY TANKS FOR DIFFERENT RATES OF SEWAGE FLOW.

Flow in sewer, in million gallons daily (1)	Ratio of storm flow to average dry-weather flow (2)	Storm flow passing tanks to sewage treatment works, in million gallons daily (3)	EXCESS STORM FLOW PASSING THROUGH TANKS		
			Excess storm flow, in million gallons daily (4)	Ratio of excess storm flow to average dry-weather flow (5)	Period of detention in storm stand-by tanks, in minutes (6)
(*a*) OLENTANGY-SCIOTO INTERCEPTING SEWER AT WHITTIER STREET. TANKS UNDER ESTIMATED 1945 CONDITIONS†					
34	1	34	0	0	....
68	2	68	0	0	....
102	3	68	34	1	180
136	4	68	68	2	90
170	5	68	102	3	60
204	6	68	136	4	45
238	7	68	170	5	36
272	8	68	204	6	30
306	9	68	238	7	25.7
340	10	68	272	8	22.5
374	11	68	306	9	20
408	12	68	340	10	18
442	13	68	374	11	16.4
487*	14.3	68	419	12.3	14.6
487*	14.3	0	487	14.3	12.6
(*b*) MAIN STREET SEWER AT ALUM CREEK TANK, UNDER ESTIMATED 1960 CONDITIONS					
6.7	1	6.7	0	0	....
13.4	2	13.4	0	0	....
20.1	3	13.4	6.7	1	180
26.8	4	13.4	13.4	2	90
33.5	5	13.4	20.1	3	60
40.2	6	13.4	26.8	4	45
46.9	7	13.4	33.5	5	36
53.6	8	13.4	40.2	6	30
60.3	9	13.4	46.9	7	25.7
67.0	10	13.4	53.6	8	22.5
73.7	11	13.4	60.3	9	20
80.4	12	13.4	67.0	10	18
87.1	13	13.4	73.7	11	16.4
93.8	14	13.4	80.4	12	15
100	15	13.4	87.1	13	13.8
134	20	13.4	121	18	10
201	30	13.4	188	28	6.4
268	40	13.4	255	38	4.7
335	50	13.4	322	48	3.8
388 *	58	13.4	375	56	3.2
388 *	58	0	388	58	3.1

* Maximum rates of flow in sewers. † Three tanks in service.

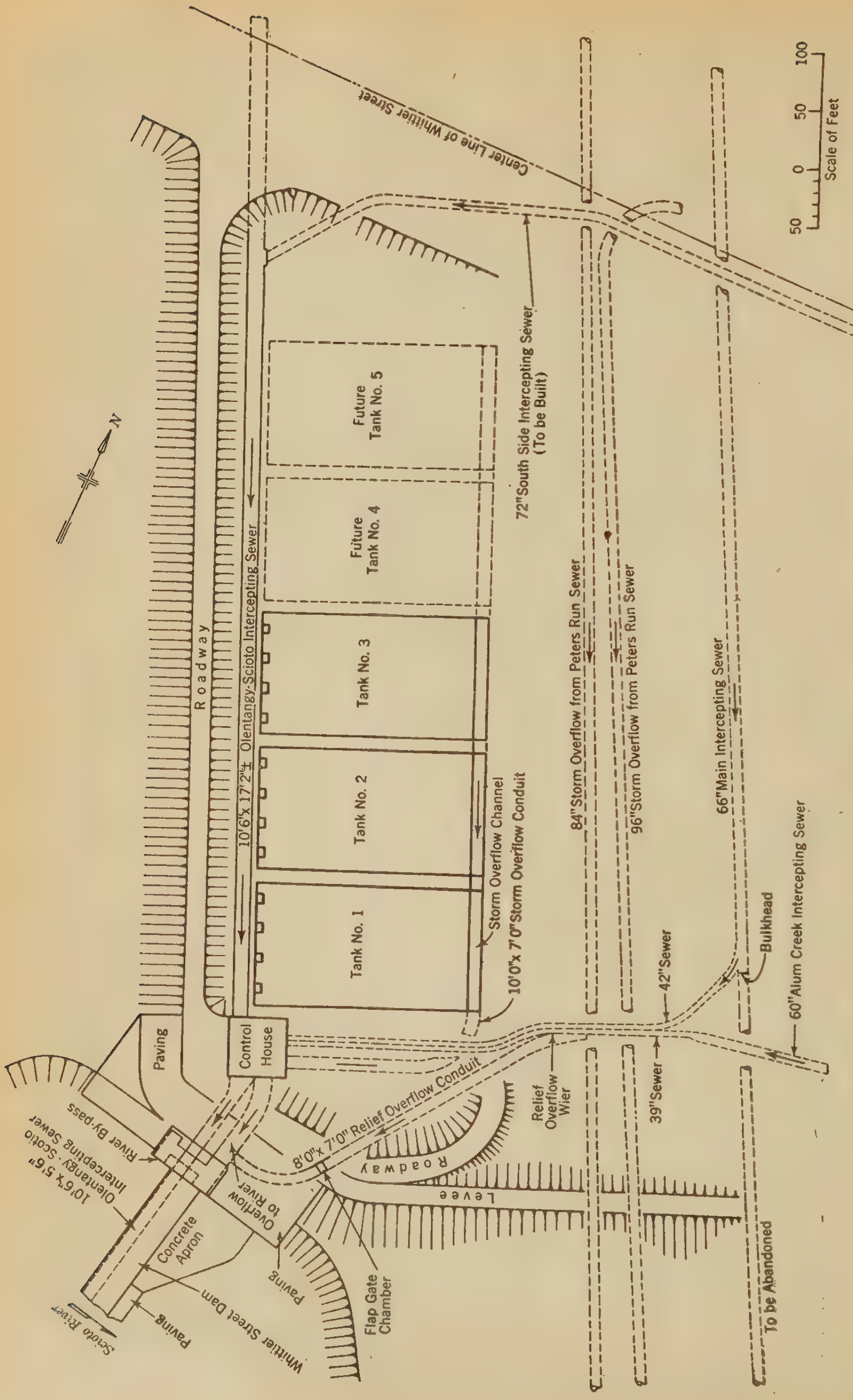
Center Line of Whittier Street
N
Future Tank No. 5
Future Tank No. 4
Tank No. 3
Tank No. 2
Tank No. 1
Roadway
10'6"x 17'2"± Olentangy-Scioto Intercepting Sewer
72"South Side Intercepting Sewer (To be Built)
84"Storm Overflow from Peters Run Sewer
96"Storm Overflow from Peters Run Sewer
66"Main Intercepting Sewer
60"Alum Creek Intercepting Sewer
Bulkhead
42"Sewer
39"Sewer
To be Abandoned
Storm Overflow Channel
10'0"x 7'0"Storm Overflow Conduit
Relief Overflow Wier
8'0"x 7'0" Relief Overflow Conduit
Control House
Paving
River By-pass
10'6"x 5'6" Olentangy-Scioto Intercepting Sewer
Concrete Apron
Overflow to River
Paving
Paving
Whittier Street Dam
Scioto River
Flap Gate Chamber
Roadway
Levee
50 0 50 100
Scale of Feet

FIG. 9.—PLAN OF STORM STAND-BY TANKS, ON WHITTIER STREET, COLUMBUS, OHIO.

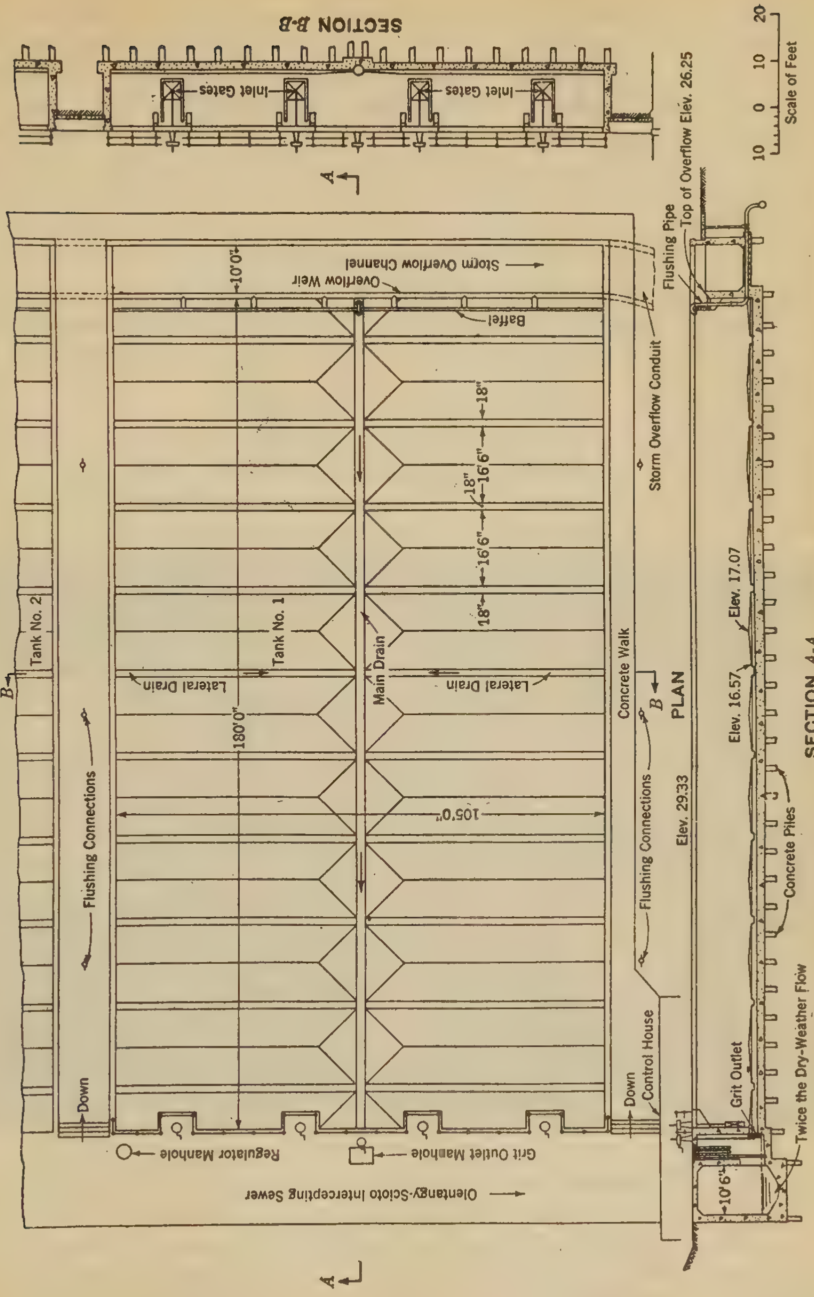
SECTION B-B
Inlet Gates
Inlet Gates
A
Storm Overflow Channel
Overflow Weir
Baffle
10'0"
18"
16'6"
18"
16'6"
18"
Tank No. 2
Tank No. 1
Main Drain
Lateral Drain
Lateral Drain
180'0"
105'0"
Flushing Connections
Flushing Connections
Storm Overflow Conduit
Concrete Walk
B
B
PLAN
Elev. 29.33
Down
Down
Control House
Regulator Manhole
Grit Outlet Manhole
Olentangy-Scioto Intercepting Sewer
A
Flushing Pipe
Top of Overflow Elev. 26.25
Elev. 17.07
Elev. 16.57
SECTION A-A
Concrete Piles
Grit Outlet
Twice the Dry-Weather Flow
10'6"
9'6"
10
0
10
20
Scale of Feet

FIG. 10.—DETAILS OF ONE OF THE STORM STAND-BY TANKS ON WHITTIER STREET, COLUMBUS, OHIO.

that the tanks should provide a detention period of 30 min when full and when the rate of flow of excess storm sewage through them is equal to six times the average dry-weather rate of flow; or, in other words, when the rate of flow in the sewer at the tanks is eight times the average dry-weather rate of flow, of which total flow twice the average dry-weather flow would go to the treatment works. The periods of detention in the Whittier Street tanks for different rates of sewage flow under estimated 1945 conditions are given in Table 4(*a*) and in the Alum Creek tank in Table 4(*b*) for estimated 1960 conditions.

WHITTIER STREET STORM STAND-BY TANKS

The storm stand-by tanks at Whittier Street, Figs. 9 and 10, consist of three uncovered reinforced concrete tanks on the east side of and immediately adjacent to the Olentangy-Scioto Intercepting Sewer. Each tank has a volumetric capacity, below the crest of the overflow weir, of 1 337 000 gal; the three tanks have a total capacity of 4 011 000 gal.

In the west wall of each tank are four 48-in. square inlet-gates and one 24-in. circular drain-gate, connecting the tank with the sewer. All these gates are operated electrically. The inverts of the openings are above the hydraulic grade that will obtain in the sewer, with a rate of flow of twice the estimated 1960 average dry-weather rate of flow, so that, at no time during normal dry-weather operation, can sewage flow into the tanks. At the east end of each tank excess storm sewage is discharged over an overflow weir into a storm overflow channel, and through a storm overflow conduit to, and through, the Control House and into the Scioto River just below the Whittier Street Dam. In front of the weir is a baffle to prevent floating solids from escaping.

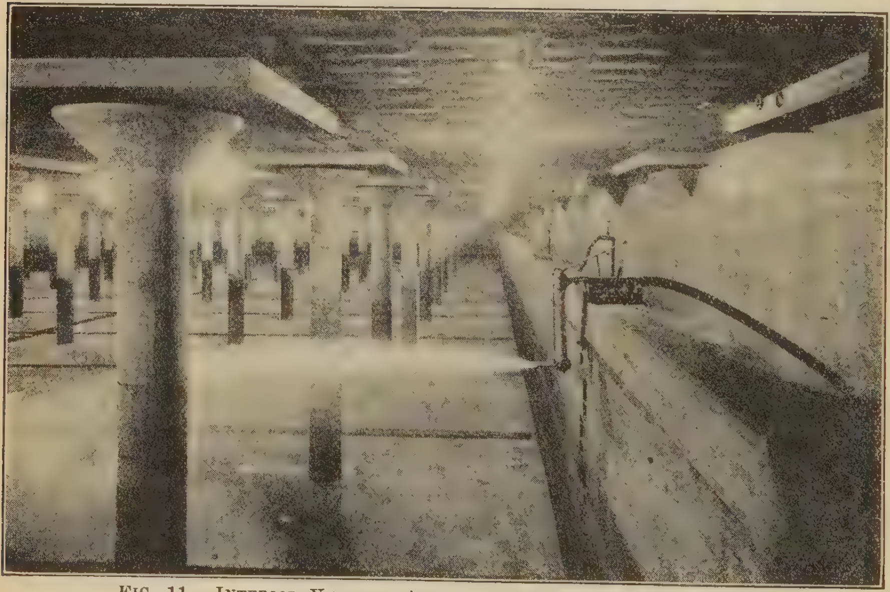

FIG. 11.—INTERIOR VIEW OF ALUM CREEK STORM STAND-BY TANK.

A movable tank cleaner (Fig. 11) is fitted to the coping of each longitudinal wall of each tank. It consists of a standard piece of fire-fighting equipment known as a "cellar pipe," mounted on a movable carriage which runs on rails set in the wall copings. This cleaner is used to flush the deposited matters back into the intercepting sewer after the tanks have been drained. At one corner of each tank is a manhole containing an electrical pressure-switch which makes an electrical contact when the level of the storm sewage in the tank reaches the elevation of the crest of the overflow weir. With high water in the river the empty tanks are subjected to upward water pressure, in excess of their weight, and it was necessary, therefore, to provide means for preventing flotation. This was done by driving pre-cast reinforced concrete holding-down piles[10] to which the tanks were anchored by extending the longitudinal reinforcement bars of the piles into the floors.

WHITTIER STREET CONTROL WORKS

When the Olentangy-Scioto Intercepting Sewer is completed, when all the lateral sewers are connected to it, and when certain small areas are converted from the combined to the separate system, the Main Intercepting Sewer will become a sanitary sewer discharging into the Olentangy-Scioto Intercepting Sewer below the Whittier Street tanks. The Alum Creek Intercepting Sewer also discharges into the Olentangy-Scioto Intercepting Sewer below the storm stand-by tanks near Whittier Street, the quantity of storm sewage entering the Alum Creek Intercepting Sewer being regulated at the Alum Creek Storm Stand-By Tank. Consequently, the rate at which storm sewage from the Olentangy-Scioto Intercepting Sewer can flow past the Whittier Street tanks and on to the treatment works will vary continuously, since it is the difference between the maximum rate of flow which the treatment works can handle and the combined rate of flow in the Main and Alum Creek Intercepting Sewers. The rate of flow in the Olentangy-Scioto Intercepting Sewer below the storm stand-by tanks near Whittier Street is regulated by locating the junction of the Main and Alum Creek Intercepting Sewers with the Olentangy-Scioto Intercepting Sewer in the Whittier Street Control House below the regulator gates on this sewer and then by controlling, automatically, the action of the regulator gates from the sewage level in this sewer below the junction.

The control works, in the Control House (Figs. 12 and 13), consist of electrically operated sluice-gates, two duplicate electric float-switches, one regulator float-switch, and a control panel, together with auxiliary electrical equipment.

Shut-Off Gates.—Three gates, 48 by 72 in., permit the shutting off of the Olentangy-Scioto Intercepting Sewer so that no sewage from this sewer or from the Main and Alum Creek Intercepting Sewers, can reach the treatment works. These gates are normally open, being closed only if the pumping station is shut down.

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[10] "Holding-Down Power of Concrete Piles—Results of Field Tests Made at Columbus, Ohio, on Square Pre-cast Piles with Parallel Sides," by John H. Gregory and Robert A. Allton, Members, Am. Soc. C. E., and James H. Blodgett, Assoc. M. Am. Soc. C. E., *Civil Engineering,* Vol. 3, February 1933, p. 66.

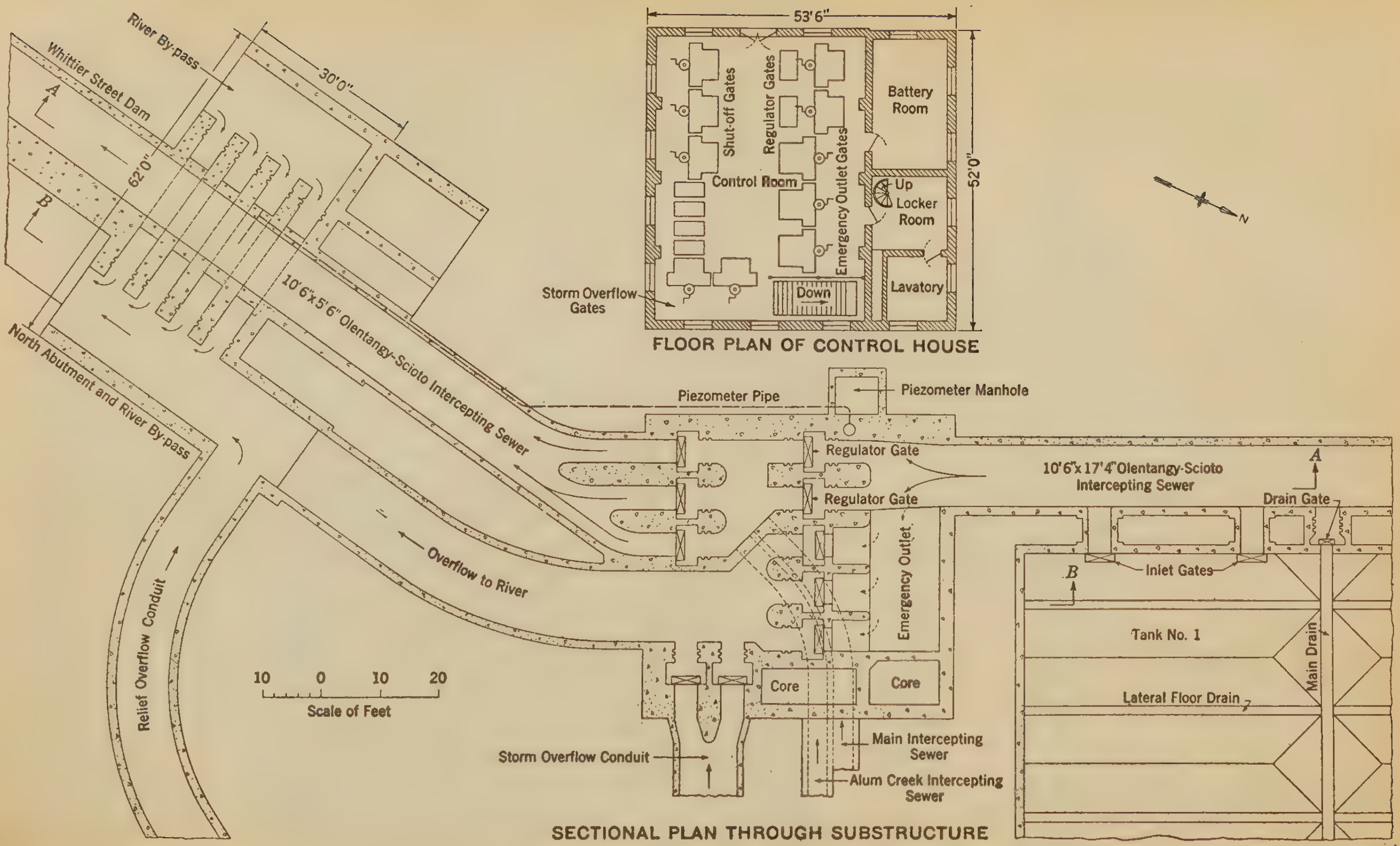
53'6"
52'0"
Battery Room
Up
Locker Room
Lavatory
Shut-off Gates
Regulator Gates
Emergency Outlet Gates
Control Room
Down
Storm Overflow Gates
FLOOR PLAN OF CONTROL HOUSE
N
River By-pass
Whittier Street Dam
30'0"
62'0"
A
B
North Abutment and River By-pass
10'6"x5'6" Olentangy-Scioto Intercepting Sewer
Piezometer Pipe
Piezometer Manhole
Regulator Gate
Regulator Gate
10'6"x17'4" Olentangy-Scioto Intercepting Sewer
A
Drain Gate
Inlet Gates
B
Tank No. 1
Main Drain
Lateral Floor Drain
Emergency Outlet
Core
Core
Overflow to River
Relief Overflow Conduit
10
0
10
20
Scale of Feet
Storm Overflow Conduit
Main Intercepting Sewer
Alum Creek Intercepting Sewer
SECTIONAL PLAN THROUGH SUBSTRUCTURE

FIG. 12.—PLANS OF CONTROL WORKS AT WHITTIER STREET STORM STAND-BY TANKS.

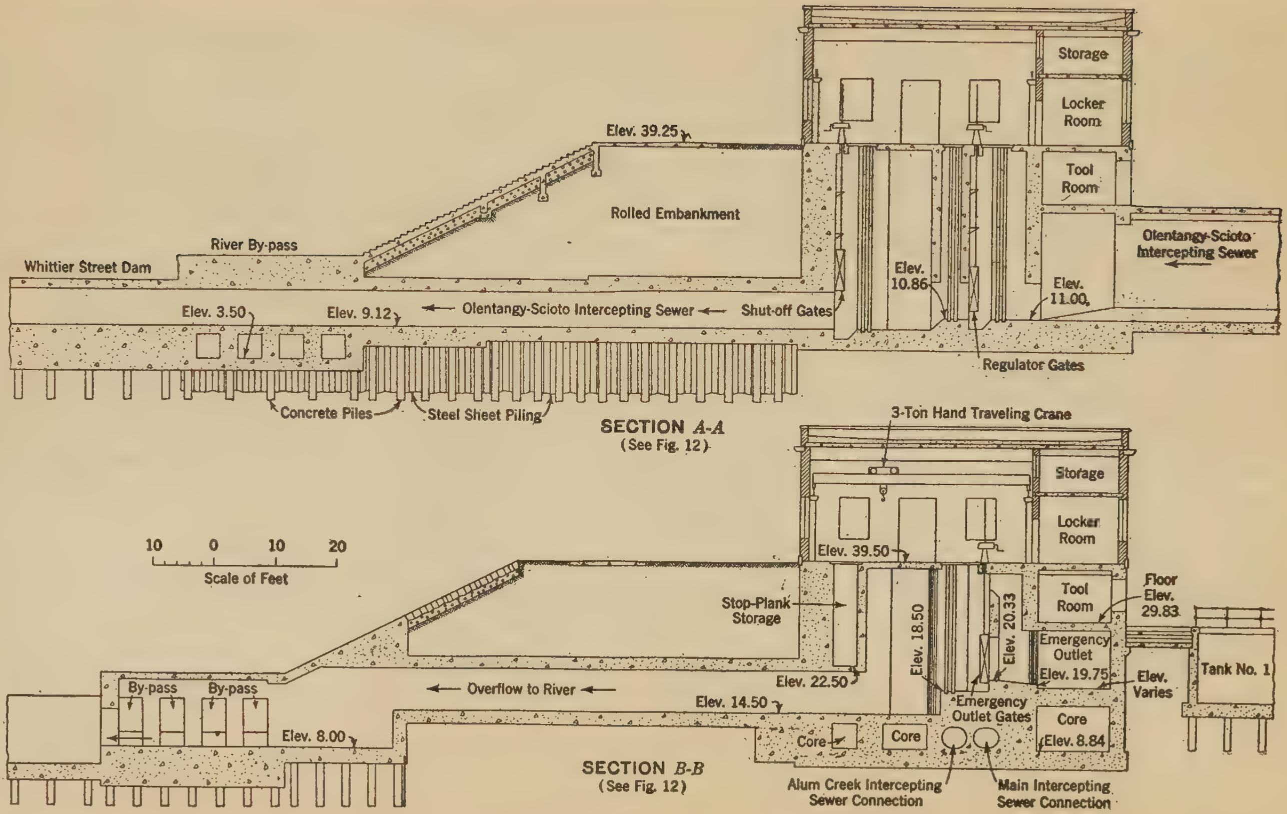
Storage
Locker Room
Tool Room
Elev. 39.25
Rolled Embankment
Olentangy-Scioto Intercepting Sewer
River By-pass
Whittier Street Dam
Elev. 10.86
Elev. 11.00
Elev. 3.50
Elev. 9.12
Olentangy-Scioto Intercepting Sewer
Shut-off Gates
Regulator Gates
Concrete Piles
Steel Sheet Piling
SECTION A-A
(See Fig. 12)
3-Ton Hand Traveling Crane
Storage
Locker Room
10
0
10
20
Scale of Feet
Elev. 39.50
Stop-Plank Storage
Elev. 18.50
Elev. 20.33
Tool Room
Floor Elev. 29.83
Emergency Outlet
Elev. 19.75
Elev. Varies
Tank No. 1
By-pass
By-pass
Overflow to River
Elev. 22.50
Elev. 14.50
Emergency Outlet Gates
Core
Elev. 8.84
Elev. 8.00
Core
Core
SECTION B-B
(See Fig. 12)
Alum Creek Intercepting Sewer Connection
Main Intercepting Sewer Connection

FIG. 13.—SECTIONS OF CONTROL WORKS AT WHITTIER STREET STORM STAND-BY TANKS.

Regulator Gates.—By means of float control, the two regulator gates, 42 by 72 in., control the volume of storm sewage passing through them from the Olentangy-Scioto Intercepting Sewer so that the rate at which storm sewage will reach the treatment works will be held to a predetermined maximum. With additional sewage treatment facilities it is planned to handle rates of flow as great as twice the average dry-weather rate of flow.

Storm-Overflow Gates.—The two storm-overflow gates, 48 by 84 in., permit shutting off the storm-overflow conduit leading from the stand-by tanks to the river so that, during periods of extreme high water when the river is at a higher elevation than the top of the tanks, it cannot back up and cause the tanks to overflow. Under normal conditions, these gates are open.

Emergency Outlet Gates.—The three emergency outlet gates, 48 by 84 in., permit the discharge of storm sewage from the Olentangy-Scioto Intercepting Sewer to the river without passing through the tanks. These gates are normally closed, and are opened only when high water in the river makes it necessary to close, not only the storm-overflow gates but, also, the inlet and drain-gates of the tanks.

Control of Gates.—Push-button controls for the inlet and drain-gates of the tanks are provided on the control panel as well as a jack and plug device for changing the sequence in which the tanks are placed in service. The terminals of the electric pressure switches at the tanks, and solenoid switches for proper control, are also provided on the control board.

Duplicate float-switches, connected in parallel, are actuated by the rise of the river and cause the closing of the inlet and drain-gates of the tanks, and, also, the closing of the storm-overflow gates, whenever the river stage exceeds Elevation 25, or about 15 in. below the crest of the overflow weirs. Other duplicate float-switches, connected in parallel, are actuated by the rise of the sewage in the Olentangy-Scioto Intercepting Sewer above the regulator gates and cause the emergency outlet gates to open when the sewage level exceeds Elevation 29.5, an elevation that cannot be reached unless the inlet and drain-gates are closed. These float-switches are installed in duplicate so as to minimize the chances of failure to operate due to the sticking of a float.

The float-switch controlling the regulator gates is actuated by the rise and fall of the sewage in the Olentangy-Scioto Intercepting Sewer below the Control House. The electrical contacts are arranged so that the gates close if the sewage level exceeds a predetermined elevation in the sewer below the gates and open if it falls 1 in. below this elevation; but they cause no movement of the gates while the sewage level is at this elevation.

All gates are operated electrically because of the freedom from difficulties due to freezing, and because of the ease of control. However, to insure operation in case of interruption of service in the power supply, all motors are for direct current and are operated from a 60-cell storage battery which is kept charged automatically by a motor-generator set.

Battery Room.—In the battery room are the switchboard, transformers, motor-generator set, and storage battery. The motor-generator set is connected in parallel with the storage battery and has a capacity of 14 amperes with a full load of 140 volts, sufficient to operate the two regulator gates without drawing on the battery. A contact on the regulator float-switch starts the motor-generator set before the regulator gates go into operation. The storage battery is of sufficient capacity to deliver 40 amperes continuously for 8 hours, with a final pressure of not less than 105 volts.

Stop-Planks.—In front of all sluice-gates are stop-plank grooves so that any gate may be isolated for repair or adjustment. The stop-planks and a stop-plank lifter are stored in grooves below the floor of the control room.

Operation During Dry Weather.—When the Olentangy-Scioto Intercepting Sewer is carrying only the dry-weather sewage flow and the river is at or below Elevation 25, the emergency outlet gates are closed, whereas the shut-off, regulator, and storm-overflow gates are open. The inlet gates and drain-gate of Tank No. 1 are open, whereas the gates on the other two tanks are closed. All the sewage in the intercepting sewer passes the tanks and flows through the regulator and shut-off gates to the pumping station and treatment works.

Operation During Storms.—Whenever a storm occurs, and the flow of sewage in the Olentangy-Scioto Intercepting Sewer increases, the level in the sewer below the Control House rises, thereby raising the float of the regulator float-switch. When the flow to the treatment works has reached the predetermined maximum rate and the level below the regulator gates is at the corresponding elevation, a contact is made in the regulator float-switch. The regulator gates then begin to close and continue to close until they throttle the flow sufficiently to cause the level in the sewer below the gates to fall, whereupon the contact breaks and the gates come to rest. If the level in the sewer below the gates now falls 1 in. below the predetermined elevation, a second contact is made in the regulator float-switch and the gates start to open, thus permitting more sewage to pass the gates and the sewage level below them to rise, until this second contact breaks and the gates again come to rest. From this time, as long as the storm continues, the gates are constantly "hunting," and maintain a level in the sewer below the gates as close to the desired elevation as possible. After the storm is over, and the flow in the intercepting sewer becomes less than the volume to be delivered to the treatment works, the second contact is made and the gates return to the open position.

As soon as the regulator gates come into action, the storm sewage in the intercepting sewer above these gates begins to back up and flow into Tank No. 1, the gates of which are open. When this tank has filled to the elevation of the overflow weir, a contact is made in the pressure-switch in Tank No. 1 which, by means of the connections on the control panel, causes the gates on Tank No. 2 to open. With the opening of the gates on Tank No. 2, this tank begins to fill, some of the storm sewage in Tank No. 1 flows back into the intercepting sewer and into Tank No. 2 until the level in the two tanks is the same, whereupon both tanks continue to fill. When both tanks are filled to the elevation of the overflow weirs, a contact is made in the pressure-switch in Tank No. 2, which, in the same manner, causes the gates on Tank No. 3 to open, and then this tank begins to fill. Some of the storm sewage in Tanks Nos. 1 and 2 flows back into the intercepting sewer and into Tank No. 3 until the level is the same in all three tanks, after which the three tanks continue to fill together. When all three tanks have been filled to the level of the overflow weirs the excess storm sewage is discharged into the storm-overflow channel and thence to the river.

This method of operation was adopted in order to prevent all the tanks from being fouled by short storms of low intensity which do not produce sufficient excess storm sewage to fill all the tanks, and thus to reduce tank cleaning to a minimum.

Since the floors of the tanks are well above the invert of the intercepting sewer, as soon as the flow in the sewer above the tanks becomes less than the flow passing the regulator gates, the storm sewage remaining in the tanks slowly drains back into the sewer until the tanks are empty. The tanks are then cleaned, using the tank cleaners on the side walls to flush out deposited solids.

Emergency and Relief Overflows.—In the Control House, over the emergency outlet gates, are three emergency overflow weirs with crests at Elevation 35 so that, if all the gates on the stand-by tanks should be closed, and the emergency outlet gates should fail to open, sewage from the Olentangy-Scioto Intercepting Sewer above the regulator gates can back up and find a free outlet to the river. On the Alum Creek Intercepting Sewer, and on the connection from the Main Intercepting Sewer to the Control House, a relief overflow weir is located, with its crest at Elevation 16.75, so that, should the shut-off gates in the Control House be closed, sewage from the Alum Creek and Main Intercepting Sewers can back up and flow over this weir into the relief overflow conduit and thence through back-water gates to the river.

ALUM CREEK STORM STAND-BY TANK

The Alum Creek Tank (Figs. 14 and 15) consists of a covered reinforced concrete tank, which, with a short length of 9-ft sewer, forms the connecting link between the Main Street Sewer and the Alum Creek Intercepting Sewer. The tank has a volumetric capacity below the crest of the overflow weir of 857 000 gal.

Along the west wall of the tank is a dry-weather flow channel which forms a continuation of the Main Street Sewer and through which sewage flows during dry weather. The elevations of the invert of this channel and of the floor of the tank are such that, for all rates of flow to twice the estimated 1960 dry-weather rate of flow, sewage will be confined to the channel and will not flow into the tank. At the east end of the tank the excess storm sewage is discharged over an overflow weir into a storm-overflow channel and through a storm-overflow conduit to Alum Creek. A baffle in front of the weir prevents floating solids from escaping. On the copings of the north and south parapet walls of the walkways are tank cleaners of the type used at the Whittier Street tanks for flushing after a storm.

Due to the fact that the tank is covered, it was felt advisable to provide positive ventilation to be used whenever workmen enter for cleaning or inspection. This was done by installing two blowers in the Control House of sufficient capacity to give one change of air every 15 min when the tank is full to the crest of the overflow weir. Fresh air from the blowers is discharged into a duct in the west wall of the tank; it enters through openings in the roof of the tank near the west wall, flows across the tank, and escapes through openings in the roof at the east end. The tank is lighted by electric lights set in water-tight marine fixtures. An interior view is shown in Fig. 11.

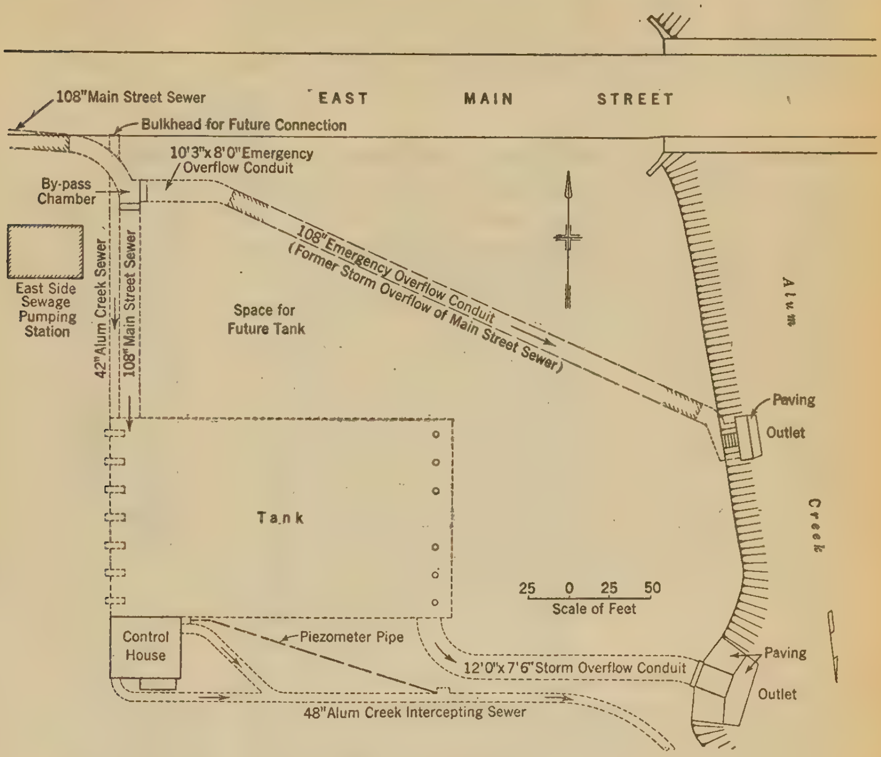
108"Main Street Sewer
EAST MAIN STREET
Bulkhead for Future Connection
10'3"x 8'0"Emergency Overflow Conduit
By-pass Chamber
East Side Sewage Pumping Station
42"Alum Creek Sewer
108"Main Street Sewer
Space for Future Tank
108"Emergency Overflow Conduit (Former Storm Overflow of Main Street Sewer)
Alum
Paving
Outlet
Creek
Tank
25 0 25 50
Scale of Feet
Control House
Piezometer Pipe
12'0"x 7'6"Storm Overflow Conduit
Paving
Outlet
48"Alum Creek Intercepting Sewer

FIG. 14.—PLAN OF ALUM CREEK STORM STAND-BY TANK, COLUMBUS, OHIO.

ALUM CREEK CONTROL WORKS

The control works consist of two electrically operated sluice-gates, a float-switch, and auxiliary electrical equipment.

Shut-Off Gate.—The shut-off gate, 48 in. square, permits shutting off the Alum Creek Intercepting Sewer so that no sewage from the Main Street Sewer can enter it. This gate is normally open.

Regulator Gate.—The regulator gate, 48 in. square, by means of a float control, automatically controls the volume of storm sewage from the Main Street Sewer passing through it so that the volume of sewage flowing in to the Intercepting Sewer is held to a predetermined maximum.

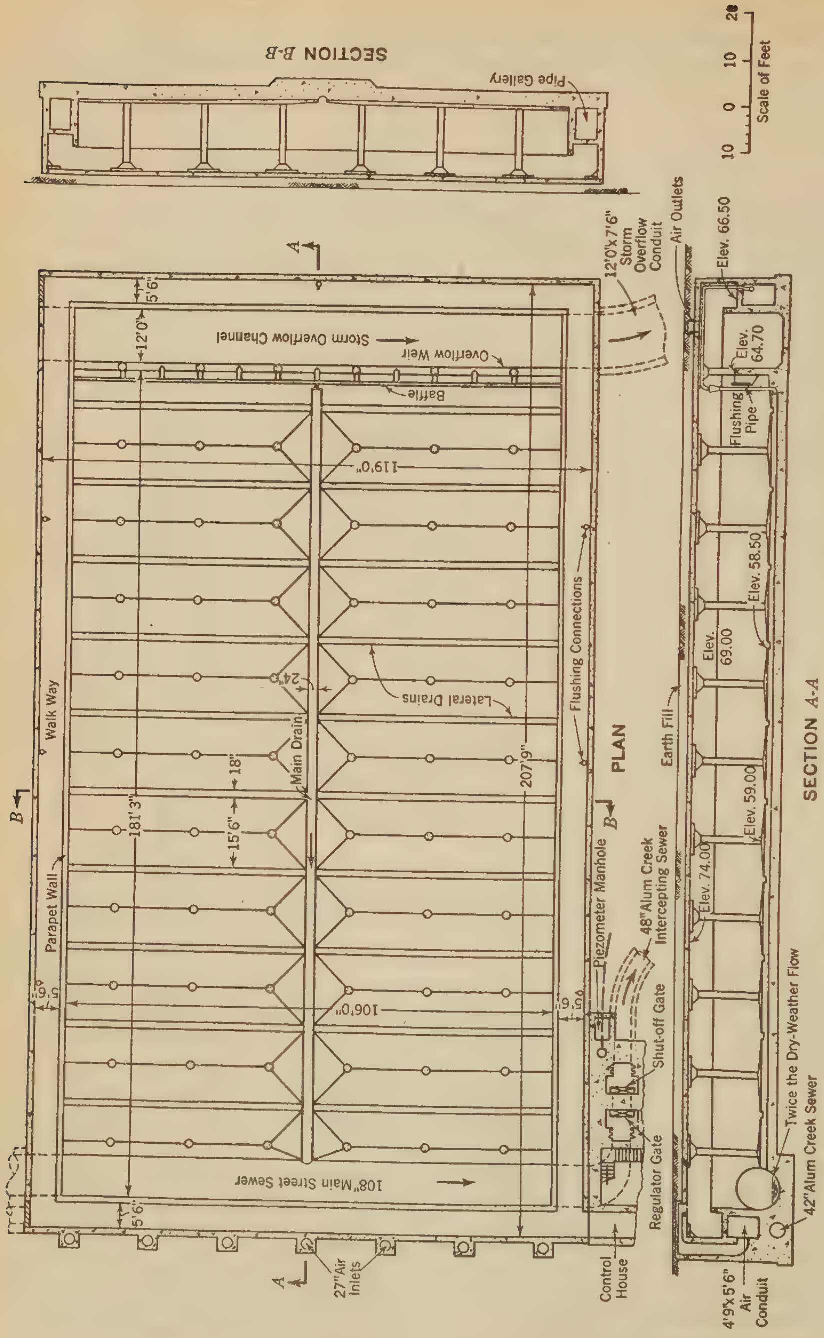
SECTION B-B
Pipe Gallery
PLAN
Storm Overflow Channel
Overflow Weir
Baffle
12'0"
5'6"
119'0"
106'0"
181'3"
207'9"
15'6"
18"
24"
Main Drain
Lateral Drains
Flushing Connections
Walk Way
Parapet Wall
5'6"
108" Main Street Sewer
27" Air Inlets
A
B
Control House
Regulator Gate
Shut-off Gate
48" Alum Creek Intercepting Sewer
Piezometer Manhole
12'0"x7'6" Storm Overflow Conduit
Air Outlets
Earth Fill
SECTION A-A
Elev. 66.50
Elev. 64.70
Flushing Pipe
Elev. 58.50
Elev. 69.00
Elev. 59.00
Elev. 74.00
Twice the Dry-Weather Flow
42" Alum Creek Sewer
4'9"x5'6" Air Conduit
10 0 10 20
Scale of Feet

FIG. 15.—DETAILS OF ALUM CREEK STORM STAND-BY TANK.

Regulator Float-Switch.—The float-switch controlling the regulator gate is actuated by the rise and fall of the sewage in the intercepting sewer below the Control House. The electrical contacts are arranged so that the gate closes if the sewage level exceeds the predetermined elevation in the intercepting sewer below the gate; it opens if the sewage level falls below, but remains at rest while the sewage is at, this elevation.

Equipment in Control House.—The two blowers are electrically driven and have a total capacity of 13 400 cu ft of free air per min. The 3-ton crane is electrically operated.

Operation During Dry Weather.—When the Main Street Sewer is carrying only the dry-weather sewage flow, the shut-off gate and the regulator gate are open, and all the sewage flows in the dry-weather flow channel at the west end of the tank, through these gates, and thence into the Alum Creek Intercepting Sewer.

Operation During Storms.—During storms the regulator gate operates in the same manner as the regulator gates at the Whittier Street tanks. When the regulator gate comes into action, storm sewage in the dry-weather flow channel backs up and flows into the tank, and, as soon as the tank is filled, flows over the overflow weir and thence to Alum Creek. It should be pointed out that the former storm over-flow at the end of the Main Street Sewer has been converted to an emergency outlet for use in case of repairs or extensions to the storm stand-by tank, and that, in times of storm, all the flow from this sewer, other than that flowing through the regulator gate, passes into the Alum Creek tank. The method of cleaning is the same as that used at the Whittier Street tanks.

COST OF WORK

Exclusive of the cost of engineering, the total construction cost of the works described was $3 637 000, as shown in Table 5, and, in Table 6, are given the costs per foot of, and the rate of progress on, the Olentangy-Scioto and Alum Creek Intercepting Sewers.

TABLE 5.—COST OF INTERCEPTING SEWERS, REGULATOR CHAMBERS, AND STORM STAND-BY TANKS.

Structure (1)	Cost	
	Sub-total (2)	Total (3)
Intercepting Sewers:		
Olentangy-Scioto	$1 670 000	....
Alum Creek	939 000	$2 609 000
Regulator chambers	....	349 000
Storm Stand-By Tanks:		
Whittier Street	$522 000	....
Alum Creek	157 000	$679 000
Total cost	....	$3 637 000

TABLE 6.—COST OF AND RATE OF PROGRESS ON OLENTANGY-SCIOTO AND ALUM CREEK INTERCEPTING SEWERS.

Type of sewer (1)	Size (2)	Length, in feet (3)	Average length of sewer completed per day, in feet (4)	Cost* per foot of completed sewer (5)	Remarks (6)
OLENTANGY-SCIOTO INTERCEPTING SEWER: RECTANGULAR					
A	10 ft. 6 in. by 5 ft. 6 in.	3 719	16	$45.09	Open cut
B	10 ft. 6 in. by 5 ft. 6 in.	590	16	57.13	Open cut
C	10 ft. 6 in. by 5 ft. 6 in.	520	5	207.93	Open cut
D	10 ft. 6 in. by 17 ft. 2 in. ±	681	4	86.02	Open cut
E	10 ft. 6 in. by 12 ft.	956	4	67.05	Open cut
F	10 ft. 6 in. by 12 ft.	250	4	87.24	Open cut
G	10 ft. 6 in. by 12 ft.	800	6	71.38	Open cut
H	10 ft. 6 in. by 12 ft.	2 334	6	80.35	Open cut
I	10 ft. 6 in. by 12 ft.	568	6	81.84	Open cut
J	10 ft. 6 in. by 12 ft.	239	5	95.88	Open cut
K	10 ft. 6 in. by 12 ft.	1 509	5	151.97	Open cut
OLENTANGY-SCIOTO INTERCEPTING SEWER: CIRCULAR					
L	11 ft. 9 in.	2 092	6†	$93.26	Tunnel
M	10 ft. 3 in.	2 813	6†	71.29	Tunnel
N	9 ft.	4 762	12	32.25	Open cut
O	9 ft.	1 251	10	63.87	Open cut
ALUM CREEK INTERCEPTING SEWER: CIRCULAR					
P	5 ft.	14 307	8†	$49.33	Tunnel
Q	5 ft.	611	4†	49.33	Tunnel
R	5 ft.	3 632	10	42.80	Open cut
S	4 ft.	3 096	26	14.41	Open cut

* Includes manholes, sub-drain, and other small miscellaneous items. † Per heading.

ENGINEERING ORGANIZATION

In the carrying through of the work described, the writers were assisted by John A. Rousculp, and Robert T. Regester, Associate Members, Am. Soc. C. E., Senior Designing Engineers, and by J. T. Moore, M. Am. Soc. C. E., and D. T. Mitchell, Harry Christiansen, and James H. Blodgett, Associate Members, Am. Soc. C. E., Field Engineers. They also wish to express their appreciation to the other members of the Engineering Organization for the real co-operation received from them at all times.

ACKNOWLEDGMENTS

The works were begun under the administration of the Hon. James J. Thomas, Mayor, and the Hon. W. H. Duffy and the Hon. R. S. McPeak, Directors of Public Service, and were completed under the present administration, the Hon. Henry W. Worley, Mayor, and the Hon. W. P. Halenkamp, Director of Public Service.

DISCUSSION

C. B. HOOVER,[11] M. AM. SOC. C. E., and C. D. MCGUIRE,[12] ESQ. (by letter). —Previous to November, 1932, the drainage of the Alum Creek Sewerage District, in Columbus, Ohio, was pumped over the Alum Creek-Scioto River divide into the Olentangy-Scioto Intercepting Sewer District for delivery to the Main Sewage Pumping Station, which, in turn, delivered this and other drainage of the city to the Sewage Treatment Works.

The pumping station, which served the Alum Creek Drainage District, was equipped with a sand-catcher, tandem coarse screens, and motor-driven horizontal centrifugal pumps which discharged into a 20-in. cast-iron force main, 8 200 ft in length. This station was in operation for twenty-three years previous to the completion of the storm stand-by tanks, during which time about 14 300 000 000 gal of sewage were pumped over the divide. Very little, if any, of this pumpage included storm drainage, and there were many interruptions of service due to floods in the creek and a lack of operating funds.

The minimum flow in Alum Creek during the warm weather season is about 0.04 cu ft per sec per 1 000 persons tributary to the district. Consequently, the wastage of sewage during storm flows constituted a nuisance menace both in theory and in fact.

Inspections of the creek below the main outfall sewer and the pumping station during the past five years indicate the condition of the water of the stream during 1929, 1930, 1931, and 1932, before the storm stand-by tanks and cross-town sewer were placed in operation, and, during 1933, when the entire flow of the District was handled by this gravity arrangement. The bio-chemical oxygen demand values of the stream water during the summer seasons of the years indicated and, as shown in Table 7(*a*), reveal the improvement accomplished by this change in the method of delivering this drainage to its intended outlet.

An inspection of Table 7(*a*) will show that the average 5-day bio-chemical oxygen demand of the stream water previous to 1933 was very much in excess of the dissolved oxygen saturation point of the water. In 1933, on the contrary, the oxygen balance was very much on the safe side; that is, there was ample dissolved oxygen in the water to satisfy the oxygen demand. The average summer-time oxygen demand of the stream water previous to 1933 was about 17 ppm, whereas, in 1933, when the mean flow of the creek was below the average for 1929 to 1932, inclusive, the oxygen demand was 2.9 ppm, or a reduction of 83% from that which obtained previously. The values in Table 7(*b*) are from records of the U. S. Geological Survey for the gauge on Alum Creek, 0.25 mile south of Livingston Avenue (see Fig. 1). The data in Table 7 (*a*) were obtained from tests taken on weekly inspections of Alum Creek.

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[11] Supt., Divs. of Water and Sewage Disposal, Columbus, Ohio.

[12] Chemist in Chg., Div. of Sewage Disposal, Columbus, Ohio

TABLE 7.—CONDITION OF ALUM CREEK BELOW MAIN OUTLET SEWER AT LIVINGSTON AVENUE, COLUMBUS, OHIO, BEFORE AND AFTER THE ALUM CREEK STORM STAND-BY TANKS WENT INTO OPERATION ON NOVEMBER 16, 1932

Month (1)	1929 (2)	1930 (3)	1931 (4)	1932 (5)	1933 (6)	Dissolved oxygen in the creek water, in parts per million, in 1933 (7)
(a) FIVE-DAY BIO-CHEMICAL OXYGEN DEMAND OF THE CREEK WATER, IN PARTS PER MILLION, AT 20° CENTIGRADE						
June	......	7.0	6.0	6.6	4.8	5.0
	......	18.0	17.0	14.0	1.1	4.7
	......	10.4	13.2	5.2	2.5	5.3
	......	14.4	13.0	......	2.8	3.7
July	......	11.0	11.0	12.0	3.2	3.4
	......	32.0	10.4	22.0	2.4	5.0
	......	22.8	19.2	16.2	3.7	2.1
	11.0	28.8	7.2	7.2	2.8	5.7
	......	34.8	8.4	......	1.8	5.4
August	12.0	27.0	6.6	8.0	2.9	6.2
	12.5	39.2	5.6	5.0	2.1	5.6
	6.3	15.4	8.0	11.2	2.6	4.2
	10.0	......	......	4.8	......	...
	11.0	......	......	......	......	...
September	20.0	18.2	3.6	5.6	2.9	2.6
	14.0	33.6	8.4	34.2	6.4	4.8
	9.0	25.2	8.8	61.0	3.8	6.7
	6.5	26.6	28.0	19.5	1.6	7.1
	......	......	26.4	......	......	...
October	11.0	47.6	49.6	13.0	3.8	8.4
	23.0	34.8	33.6	36.0	2.2	7.2
	21.0	21.6	28.0	2.4	0.8	7.3
	12.6	23.8	12.0	10.5	......	...
	11.4	16.8	......	......	......	...
Sums	191.3	509.0	324.0	294.4	54.2	100.4
Means	12.8	24.2	15.4	15.5	2.9	5.3
(b) STREAM FLOW FOR THE PERIOD, JUNE TO OCTOBER, INCLUSIVE, IN CUBIC FEET PER SECOND						
Mean	67.8	9.1	62.0	24.9	15.2	...
Maximum	1240	42	1220	634	150	...
Minimum	7.3	3.4	2.4	2.5	3.5	...

JULIAN MONTGOMERY,[13] M. AM. SOC. C. E. (by letter).—Before sewage treatment works were considered necessary to prevent the pollution of streams, combined sewerage systems were considered practical and economical. Since 1900 probably not more than 1% of the sewerage systems constructed, have been combined systems. To-day, practically all State Health Boards require sewerage systems to be designed on the separate plan.

It is interesting to note that sewers first were constructed at Columbus, Ohio, in 1853, eighty years ago, and that they are functioning very well to-day. It is not surprising, furthermore, to note that the present plans call for using separate systems wherever possible.

In an experience of twenty years the writer has never designed a combined system. On several occasions he has been forced, by reasons of sanitation and economics, to reduce grades below the minimum recommended. Ordinarily, it has been the custom to use $n = 0.015$ for Kutter's formula in designing sewers; whereas for the pipe manufactured during the last few years the use of $n = 0.013$ would more nearly approach the true conditions. It is the writer's experience that by using care in the inspection and laying of pipe, flatter grades than those usually recommended, can be used without inconvenience.

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[13] Cons. Engr. (Montgomery & Ward), Wichita Falls, Tex.

In designing sanitary sewerage systems at Austin and Wichita Falls, Tex., 125 gal per capita daily was the value assumed. In 1916, weir measurements of the sewage flow at Austin gave an average daily flow of 55 gal per capita contributing. In 1925, weir measurements of the sewage flow at Wichita Falls gave an average daily flow of 60 gal per capita contributing.

To-day, the writer assumes for the design of sewerage systems for cities with populations of less than 7 500 an average of 60 gal daily per capita contributing, which also includes infiltration. It should be noted that for cities in the semi-arid districts, exfiltration deserves more consideration than infiltration. The idea at all times, of course, is to construct water-tight joints.

The design of both storm and sanitary sewerage systems calls for various assumptions, all of which affect the size of the mains and laterals, and, accordingly, the cost of the system. Not always should the same assumptions be made for the small city that are made for the larger cities in the design of a sanitary sewerage system. If this is done, dead capital will be sometimes invested, and the resulting system will produce a septic effluent.

A study of a complete plan for the given city results in the making of logical design assumptions. When a city has been zoned into its residential, commercial, and industrial districts, when the main highway arteries have been laid out, and when the other utility systems have been designed logically for a reasonable period into the future, then the more logically and accurately can the assumptions be made for designing the storm and sanitary sewerage systems.

The inclusion of storm stand-by tanks in the plans for the Columbus System presents a new field for thought and study. The authors evidently took every practical condition into consideration in the design. Their preliminary studies may form the nucleus for a future treatise on the subject. The cost of such structures, of course, must always be balanced against the benefits to be derived.

D. T. MITCHELL,[14] ASSOC. M. AM. SOC. C. E. (by letter).—Little space has been alloted in this paper to description of the actual construction. Having been associated with the authors on the construction of the intercepting sewers and regulator chambers, the writer feels that a discussion of this phase will be of interest.

For the majority of the intercepting sewers and regulator chambers the concrete was mixed for 3 min in 1-cu yd mixers of the paving type. For sewers, Types *L* and *M* (Fig. 5), and the Broad Street and Chestnut Street Regulator Chambers, however, truck mixers were used, while for Type *R* sewer (Fig. 8), and for a part of Types *A* and *P*, the concrete was mixed at a central mixing plant from which it was transported, in trucks, equipped with mechanical agitators, to the site of the work. Where truck mixers were used, the concrete was mixed for 5 min.

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[14] Field Engr., Sewer Dept., Div. of Eng. and Constr., City of Columbus, Columbus, Ohio.

The mix was transported to the forms by buckets, buggies, or cars. On some of the work, a pipe held in a vertical or substantially vertical position, which was kept full at all times during the placing of concrete, was used. Such a pipe, however, was not permitted to discharge directly into the forms, but was used to transport the concrete from the mixer to a hopper or to the buggies or cars. Individual chutes longer than 8 ft, or a system of chutes the aggregate length of which was more than 8 ft, were not permitted.

TABLE 8.—UNIT CONTRACT PRICES FOR THE PRINCIPAL ITEMS.

Type (1)	Year (2)	Excavation per cubic yard (3)	Timber piles per foot of pile (4)	Steel sheet-piling per square foot of wall (5)	Reinforcing steel per pound (6)	Concrete per cubic yard (7)	Vitrified clay liner plates per square foot (8)	Miscellaneous cast-iron and steel per pound (9)
(*a*) OLENTANGY-SCIOTO INTERCEPTING SEWER								
A.....	1932	$1.20	....	....	$0.03	$9.00	0.20	$0.05
	1930	1.20	....	....	0.035	10.85	0.183	0.12
B.......	1930	1.20	....	....	0.035	10.85	0.183**	0.12
C.....	1930	1.20	$0.60	$1.25	0.035	10.85	0.183**	0.12
	1930	0.45–0.50*	0.60‡	1.25	0.0305	10.00	0.18	0.08
D.......	1930	0.45–0.50*	2.10‡	....	0.0305	10.00	0.18	0.08
E.......	1930	1.05	....	....	0.035	10.50	0.188**	....
F.......	1930	1.05	....	....	0.035	10.50	0.188**	....
G.......	1928	1.75	....	....	0.0375	11.50	0.204**	....
H.......	1928	1.75	....	....	0.0375	11.50	0.204**	....
I.......	1928	1.75	....	....	0.0375	11.50	0.204**	....
J.......	1929	1.20–1.30*	0.51	0.037§	0.04	11.50	0.188**	0.15
K.......	1929	1.20–1.30*	0.51	0.037§	0.04	11.50	0.188**	0.15
L.......	1930	8.38	....	....	0.04	14.00	0.20	0.13
M.......	1930	8.38	....	....	0.04	14.00	0.20	0.13
N.......	1932	1.40	....	....	0.04	6.26	0.15	....
O.......	1932	1.40	....	....	0.04	6.26	0.15	....
(*b*) ALUM CREEK INTERCEPTING SEWER								
P.......	1929	$21.00†	....	....	$2.60†	$18.00‖	....	....
Q.......	1929	21.00†	....	....	....	18.00‖	....	....
R.......	1929	....	....	....	....	42.00¶	....	....
S.......	1930	1.00	....	....	....	6.75	$5.50††	....
(*c*) REGULATOR CHAMBERS								
Broad Street.	1930	$5.62	....	....	$0.04	$21.25	$0.20	$0.13
Chestnut St..								
Peters Run...	1931	2.40	$2.00	....	$0.038	$12.40	$0.20	$0.15
Rich Street...								
Town Street..								
Henry Street.								
First Avenue...	1932	1.20	....	....	0.04	11.00	0.15	0.04

* Wet excavation. † Price per linear foot of sewer. ‡ Concrete piles. § Per pound. ‖ Price per linear foot of sewer, including liner plates. ¶Total price for complete sewer per linear foot. ** Computed from prices per linear foot of sewer. †† Price per linear foot of sewer for segment block.

The concrete for sewers, Type *D* and part of Types *A* and *C* (Fig. 4), was made in the proportions of 1:5.5; that for Types *N* and *O* (Fig. 5), in which high early strength to permit back-filling in 4 days was desired, in the proportions of 1 volume of cement to 4 volumes of aggregate; and that for the cradle and backing under and around the vitrified clay segmental block sewer of Type *S* (Fig. 8), in the proportion of 1 volume of cement to 12 volumes of aggregate. For the other types of sewer and for the Broad Street and Chestnut Street Regulator Chambers, the concrete was made in the proportions of 1 volume of cement to 6 volumes of aggregate; and for the other regulator chambers, in the proportion of 1 volume of cement to 5.5 volumes of aggregate. On Types *N* and *O*, an average 4-day compressive strength of 2 700 lb per sq in. was obtained.

Throughout the work, special attention was directed toward the making of concrete of uniform quality. In 1931, data pertaining to the breaking of 1 051 test cylinders,[15] representing 34 127 cu yd of concrete made in the proportions of 1 volume of cement to 6 volumes of aggregate, were tabulated. The average breaking strength at 28 days of the 1 051 cylinders was 3 371 lb per sq in., the average for cylinders testing less than that amount was 2 901 lb per sq in., and the average for cylinders testing more was 3 877. Of the 1 051 cylinders, 52% tested less than the average strength and only 2% tested less than 2 000 lb per sq in. Comparable results were obtained on the remainder of the work.

Ordinarily, wall forms for the concrete in the rectangular sewers and regulator chambers were held in place 4 days, while those for the roof slabs in the rectangular sewers were held for 6 days, and those for the regulator chambers, 7 days. The invert and arch forms for Types *L* and *M* were held 3 days, and those for Types *P* and *Q*, 1 day. For Types *N* and *O*, the invert forms were held 15 hr and the arch forms, 2 days, while on Type *R* the invert form was held 20 hr and that for the arch, 2 days.

Vitrified clay liner plates in the arches of the circular sewers and in the walls and roofs of the rectangular sewers were attached to the forms and the concrete was placed against them. In the lower part of the circular sewers, and in the bottom of the rectangular sewers, the plates were laid in mortar after the concrete had been placed.

The writer feels that cost data in more detail than those given by the authors are of interest and, therefore, has included, in Table 8, the unit prices for the principal items involved in the construction of the intercepting sewers and regulator chambers.

Robert Spurr Weston,[16] M. Am. Soc. C. E. (by letter).—The sanitary engineering history of Columbus, Ohio, is a most interesting one, especially to the writer, whose first visit was made more than thirty years ago when the late Rudolph Hering, M. Am. Soc. C. E., was reviewing the earlier work of John W. Alvord, M. Am. Soc. C. E., and the late Julian Griggs, M. Am. Soc. C. E. In those days prevailing winds brought offensive odors to the heart of the city from the putrefying pools in the Scioto River just below, and the typhoid fever death rate was sometimes of epidemic degree.

The problems of Columbus are those of a large and growing city situated on small rivers where, as elsewhere, there is a strong trend away from excessive concentration of population in the municipal center, and also a tendency to extend, by annexation and other arrangements, the muncipal utilities to surrounding suburban areas. Columbus is typical in this respect, and the authorities have acted wisely in designing for populations in 1960 which, at this time, hardly seem realizable.

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[15] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1337.

[16] Cons. Engr. (Weston & Sampson), Boston, Mass.

Interesting as are the intercepting sewers and the carefully designed control works, the stand-by tanks are of greatest interest because new in practice.

Those who study rivers and the condition of river water, are beginning to realize how easy it is for deposits to form near the mouths of sewers, especially combined sewers, even in considerable streams; how the sand and silt deposited from sewage entrains with it putrescible organic matter; how little is the cleansing effect of scour in some rivers; how much reliance must be placed upon the self-purification of a stream by digestion; and how foul a stream becomes when its digestive capacity is exceeded. With these facts in mind, even if no economies in construction were effected, the installation of stand-by tanks is commendable.

Were the flow curves for overflowing sewage and receiving river water similar, one might rely more confidently upon the transportation of suspended matter discharged from storm overflows to places of safe disposal down stream, but frequently during storms, the maximal discharge of suspended matter from the sewers occurs before the river has risen materially. Then, deposits occur which, once formed, are less easily transported down stream than is the component suspended matter before its deposit.

The stand-by basins also serve as emergency additions to the treatment works, by preventing excessive rates through the subsiding basins, and by feeding more gradually the load of suspended matter. On the other hand, is it not probable that the bulk of true domestic sewage solids is brought to the treatment works with the flows which the intercepting sewer will carry, and that the solids stored in the stand-by basins are largely those derived from the storm sewage and, therefore, are different in character?

W. W. Horner,[17] M. Am. Soc. C. E. (by letter).—Apparently, because of the removal of the normal dry-weather flow from the Scioto River, the problem at Columbus, Ohio, approached quite nearly the drainage situation of those residential communities commonly traversed by smaller watercourses. To this extent, the situation has much in common with the sanitary and drainage problems now being encountered in connection with the improvement of the suburbs of some of the larger cities in this country, such, for example, as Westchester County, New York, sewerage, the suburban situation around Philadelphia, Pa., and Washington, D. C., and, more recently, the studies that the writer has had to make in connection with the proposed sewerage in St. Louis County, Missouri.

In many of these cases, the existence of combined sewers, or the belief that storm water may enter through cellarway drains or occasional downspout connections, makes it essential that storm drainage, to some extent, be allowed for in the main trunk or intercepting sewers. Because of this, these intercepting sewers might equally well be classified as deficient storm sewers and in common with the engineers in Columbus the writer has found it necessary to design on that basis.

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[17] Cons. Engr., St. Louis, Mo.

In one large district a design was made on the basis of Fig. 16 for the preparation of which it was assumed that 50% of the houses would have downspouts connected to the sewers and that the gutter system for each house would be more than 50% efficient; actually, it was not expected that so many downspouts would become connected against regulations, but it was regarded as certain that many of the houses would have bell-traps at the foot of the outside cellar stairs, and that an appreciable quantity of yard drainage would enter the system in that way.

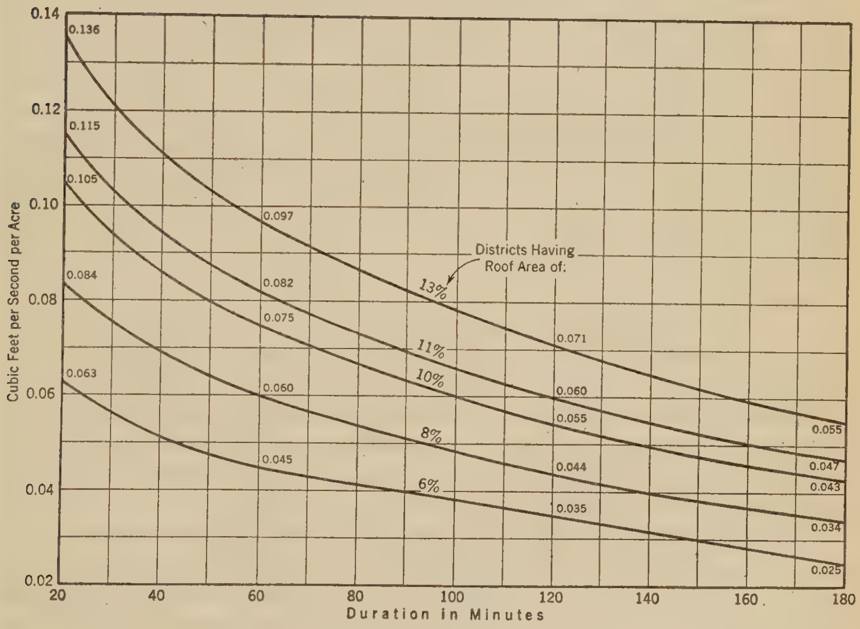

FIG. 16.—SANITARY RUN-OFF CURVES USED IN DESIGN OF SEWERS IN ST, LOUIS COUNTY, MISSOURI.

The design was completed on this basis for the full lateral trunk system of a 10 000-acre district. In this instance, while it was proposed to permit overflow to the creek in one or two places, it was believed that the design was actually balanced and that no overflow would take place. In another instance, the laterals were designed for a similar curve, but the intercepting, or more correctly, the main trunk, sewer paralleling the principal stream was designed on a basis of the maximum sewage flow being four times the average. Another condition was that this peak flow would be diluted with storm water on a 6 to 1 basis throughout the upper reaches, and on a 4 to 1 basis where it paralleled a larger stream before overflow occurred. In the latter case, a frequency of overflow was not determined accurately, but the probabilities would lie somewhere between four and six times per year. In this instance, the local stream was a small one and the dry-weather flow, exclusive of sewage, probably did not exceed 1 cu ft per sec.

In a third case, existing lateral sewers were laid out on a combined basis, and intercepting sewers were designed for six dilutions of the dry-weather flow without regulators. In general, the basis for sewer design in these districts fell into the following groups (the units being given in cubic feet per second, and the areas averaging 10 to 15 people per acre): (*a*) Sanitary sewers, designed for peak flow, about four times the average at 0.007; (*b*) intercepting sewers, carrying four to six dilutions of storm water, about 0.04; (*c*) design for some downspout connections (see Fig. 16), depending on time of flow, 0.10 to 0.05; and (*d*) full storm-water design, depending on time of flow, 1.82 to 0.4. It appears that the Olentangy-Scioto Sewer carries six or seven dilutions of storm water.

The writer notes that, in computing storm-water run-off, the coefficient of 35% for rainfall is used for residential districts. Gaugings at St. Louis, Mo., indicate, in one residential block of fairly good slopes, a coefficient averaging 0.6 and, in one of much flatter slopes, a coefficient averaging slightly less than 0.4. For a semi-commercial block, the St. Louis coefficients will average slightly more than 0.8 as compared with the 0.75, the latter being a very satisfactory agreement.

In discussing this paper with other engineers, the question has been raised as to whether an estimate was prepared for a complete new system of sanitary sewers for the area now served by combined sewers, leaving the latter to act as storm sewers only, and whether the cost of such a new system might have been, in any way, comparable with the alternatives actually adopted. It would also be interesting to know whether such a suggestion might have been impracticable because of a fixed habit of connecting downspouts to the house sewer system.

The various types of sewer structures to meet particular conditions are unusually interesting, but an explanation of the reason that led to the use of vitrified clay liner-plates (particularly for their use in the inverts of some sections and for the full circle in certain others), would be a valuable addition to the paper.

In a recent St. Louis project, involving a depth of flow varying from one-half full to full, for the handling of sewage, vitrified clay liners were used for the upper half of the circle exposed to condensation, but a high-grade concrete was relied on for the lower part where the flow was continual.

The use of the storm stand-by tanks offers an ingenious solution, but the economics of their injection into the design have not been presented as fully as the writer would have wished. It would be interesting to know the estimated cost of extending the intercepting sewer, full size, to the disposal plant, and whether somewhat less elaborate detention basins might not have been used at that location.

Samuel A. Greeley,[18] M. Am. Soc. C. E. (by letter).—This excellent paper gives much useful information relating to the difficult problem of the disposal of storm sewage from combined sewers. The long experience recorded, 1908 to 1933, is unusual and adds greatly to the value of the paper.

Intercepting sewers serving combined sewers are not built large enough to carry the entire flow of the tributary sewers, which occurs infrequently at times of heavy rains, but are generally designed to carry several times the dry-weather flow. The unit capacity of such an intercepting sewer for any particular location is thus a matter of engineering judgment which must be guided by experience with operating intercepting sewers. The authors, long familiar with the sewerage of Columbus, Ohio, have selected methods and unit capacities based on observed conditions.

It is stated that the limited capacity, and size, of connections to the old Main Intercepting Sewer resulted in about thirty-five overflows of storm sewage per year from each of the combined sewers and that its capacity was barely sufficient to handle the average dry-weather flow. It would be of interest to know about what unit capacity this old sewer had at the time when relief was provided. Assuming that near its lower end it served 150 000 persons, or about one-half the 1930 population, and had a capacity of 45 000 000 gal daily (72 in. in diameter and a velocity of 2.5 ft per sec), the unit capacity is 300 gal per capita per day. The actual unit capacity undoubtedly varied for different sections of the sewer and with the deposits, which were removed only at intervals. In providing relief the section serving combined sewers was given an average unit capacity of about 1 900 gal per capita daily.

This, of course, is relatively high, as many intercepting sewers have been built with unit capacities of 350 to, perhaps, 550 gal per capita daily. However, one section of the intercepting sewer system of Springfield, Ill.,[19] 39 000 ft long, was designed with an average unit capacity of 2 125 gal per capita daily. This relatively large capacity was to protect an impounding reservoir to be built later. The capacity of the intercepting sewer below this large section was 300 gal per capita daily, and an outlet was provided so that the excess storm sewage could be settled and chlorinated when required.

The relating of these unit capacities to rainfall intensities is of interest. It should be noted that a rainfall rate of 0.45 in. per hr, an average run-off coefficient of 0.4, and a population density of 25 per acre results in a storm sewage flow at the rate of 4 670 gal per capita daily, and that this rate, for a rainfall duration of 60 min, may occur four times per year (see Fig. 2). This problem was studied by the writer in connection with the sewage disposal of Minneapolis and St. Paul, Minn., where the unit capacities of intercepting sewers were related to a rainfall rate of 0.04 in. per hr. Studies of the rainfall indicated that, on this basis, storm sewage overflows would occur for about 130 to 150 hr per yr. Both the frequency and duration of storm sewage discharges are pertinent, and it is hoped that more data such as are given in this paper will be forthcoming. The authors concluded that storm sewage overflows of four to eight times per year would be acceptable for the central part of Columbus.

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[18] Hydr. and San. Engr. (Pearse, Greeley & Hansen), Chicago, Ill.

[19] *Engineering News-Record*, December 22, 1927.

Below the central part of the city, the capacity of the intercepting sewer is reduced, and the overflow is passed through storm stand-by tanks with displacement periods at Whittier Street, ranging from 12.6 to 180 min. As far as the writer knows, these are the first storm stand-by tanks to be built at any considerable distance from main sewage treatment works. The procedure effects a reduction in cost by limiting the relatively large intercepting sewer to that part of the stream needing such protection. Several storm sewage settling tanks have been built at main treatment works as, for instance, those for Decatur, Ill., designed in 1931. Such installations indicate the rise in the desired standards of stream cleanness and, therefore, are significant.

It would be of interest to have the unit capacities of the old and new intercepting sewers stated by the authors as an illustration of the unit capacity at which relief was required under local conditions at Columbus.

JOHN H. GREGORY,[20] ORRIS BONNEY,[21] AND ROBERT A. ALLTON,[22] MEMBERS, AM. SOC. C. E. (by letter).—Referring to the discussion by Messrs. Hoover and McGuire, they clearly bring out, by the data in Table 7, the greatly improved condition of Alum Creek at, and just below, Main Street, as contrasted with the objectionable conditions of previous years with which the writers, as well as Messrs. Hoover and McGuire, are familiar. The marked improvement in the creek, since the works were built, is a source of great satisfaction to all concerned. From an analytical standpoint, and also from visual observation, Alum Creek within the city is now typical of a clean Ohio stream.

Mr. Montgomery refers to the policy of the City, since 1923, of building new sewers on the separate system wherever possible. From the standpoint of river and stream cleansing, much is to be said for the use of the separate system, particularly with low stream flows, as is the case during certain periods of the year in the rivers and streams at Columbus. In fact, under certain conditions, where low stream flows obtain, it is possible that it might prove desirable in the interests of stream cleansing, not only to build new sewers on the separate system, but to go so far as to convert an existing combined system to a separate system, into the consideration of which would enter, of course, the elements of economy and feasibility.

As to one of the points raised by Mr. Weston, it is not only probable but a fact that the bulk of true domestic solids will be taken to the treatment works with the flows which the Intercepting Sewer will carry. The objectionable conditions in the rivers and streams are due not only to these solids, but to the escape of sewage solids and street washings at times of storm overflow, and it was to reduce the quantity of these escaping sewage solids and street washings during storm overflows that the storm stand-by tanks were built. As to the character of the solids retained in the storm stand-by tanks after a storm—these solids have, in general, the appearance of raw sludge removed from preliminary settling tanks. However, there is a very distinct classification of the solids deposited in a tank, those near the inlet end being noticeably sandy. For this reason, it would be extremely difficult to obtain a sample that would be truly representative of the sludge retained in the tanks and, hence, no analyses of the deposited solids have, as yet, been made.

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[20] Cons. Engr.; Prof. of Civ. and San. Eng., The Johns Hopkins Univ., Baltimore, Md.
[21] Sewerage Relief Engr., Div. of Eng., Dept. of Public Service, Columbus, Ohio.
[22] Sewage Disposal Engr., Dept. of Public Service, Columbus, Ohio.

As to the point raised by Mr. Horner, relative to the construction of a complete new system of sanitary sewers for the areas now served by combined sewers, leaving the latter to act as storm sewers only, it should be stated that the desirability of such a plan from the standpoint of river and stream cleansing, was not lost sight of, but it was not considered feasible under Columbus conditions to build a complete new system of sanitary sewers because the streets and alleys in which these sewers would have been located are for the most part built up and to a large extent paved. Such a plan would have involved, in the case of each building, the divorcing of the storm water from the sanitary sewage, the construction of a separate service to the street or alley either for sewage or storm water, as well as the revision, in an unknown number of cases, of the plumbing within the building. In the absence of an actual count, but assuming five persons per building and an estimated population, in areas served by the combined system, of 224 000, there would be approximately 45 000 buildings where such revision would have had to be made in order to change from the combined to the separate system. The Main Intercepting Sewer has been found to be of inadequate capacity under present conditions, and a new intercepting sewer or a new trunk sewer would have been required. Hence, the only possible saving in cost would have resulted from the construction of an intercepting sewer or trunk sewer of lesser capacity than the one actually built. Under Columbus conditions, it is believed that the cost of building a complete new system of sanitary sewers and sewer services would have exceeded whatever saving might have been realized by the construction of an intercepting sewer of lesser capacity and the omission of the storm stand-by tanks. In a few small areas in the Intercepting Sewer and East Side Districts, however, the plans do call for new sanitary sewers with the existing combined sewers to serve as storm sewers only. Elevations and the elimination of certain storm overflows were controlling elements in the adoption of this plan in these areas.

Mr. Horner has also asked relative to the reasoning that led to the use of vitrified clay liner plates. Generally speaking, the use of clay liner plates was adopted as a means of extending the useful life of the sewers as much as possible. Where the sewers were designed to flow full with the estimated maximum rate of dry-weather flow for 1960, the entire inner surface was lined. Where the sewers were designed to flow full (or nearly full) with a volume of flow made up of the estimated maximum rate of dry-weather flow for 1960 plus a considerably larger volume of storm water, only the lower parts were lined, the linings having been carried up to elevations about 1 ft above the surface of the sewage corresponding to the maximum rates of dry-weather flow for which the sewers were designed.

As to the other point raised by Mr. Horner, that is, the extension of the Olentangy-Scioto Intercepting Sewer, full size, to the sewage treatment works —such an extension, while possible, would have cost at least $200 000 more than the sewer as planned. Again, the storm stand-by tanks would have been of substantially the same design and would have cost fully as much as the tanks as built. Furthermore, the arrangement of the new sewage treatment works, on land owned by the City, did not readily lend itself to the construction of the storm stand-by tanks at that location, except by the purchase of additional property. Under existing conditions it was more economical, therefore, to locate the tanks at Whittier Street where the City owned sufficient property on which to build them than at the site of the sewage treatment works. Furthermore, at Whittier Street, the last combined sewer will have been intercepted and connected into the Olentangy-Scioto Intercepting Sewer, and there seemed to be no reason why the overflow from the storm stand-by tanks could not be discharged at this point.

Referring to Mr. Greeley's discussion, the unit capacity of the Main Intercepting Sewer, based on the 1930 populations tributary to this sewer, varied from a minimum of 150 gal to a maximum of 330 gal per capita daily, over a length of about 7 miles. In one stretch of the sewer, about 1 mile in length, the capacity is approximately 900 gal per capita daily. Obviously, over about seven-eighths of the length of the sewer, relief was needed. The capacity of the Olentangy-Scioto Intercepting Sewer, south of Hudson Street (Table 3), varies from 600 to 2 110 gal, with an average of about 1 900 gal per capita daily. Bearing in mind the very low stream flows in the Olentangy and Scioto Rivers during the warmer months of the year, these relatively large capacities were provided in order to limit, as far as feasible, overflows of storm sewage from the combined sewers tributary to these rivers.

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DEFORMATION OF STEEL REINFORCEMENT DURING AND AFTER CONSTRUCTION

By Sergius I. Sergev,[1] Esq.

With Discussion by Messrs. William Russell Davis, J. R. Worcester, F. E. Richart, Homer M. Hadley, F. N. Menefee, and Sergius I. Sergev.

Synopsis

The object of this paper is to compare, during and after construction, observed unit stresses in longitudinal steel reinforcement in helically reinforced concrete columns and **T**-beams with calculated theoretical unit stresses and with allowed unit stresses.

Three columns of high strength concrete, and three columns and six **T**-beams of average strength concrete were under observation. The calculated theoretical unit stresses due to loads were obtained, neglecting the effect of the helical reinforcement and employing ratios of the modulus of elasticity of the steel to the modulus of elasticity of the concrete ordinarily used in designs. No claim is made for the absolute accuracy of these stresses.

The allowed unit stresses as recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete[2] are empirical. A comparison of these stresses with the observed stresses indicates the departure of the actual from the anticipated. Such a comparison also shows the validity of using allowed unit stress as an indicator of the actual unit stress prevailing in the member.

This paper is presented not as a completed study on the subject of shrinkage, but as a forerunner for future investigations.

Introduction

Strength designs for reinforced concrete, like those for other materials of construction, are based upon supposedly rational assumptions from which

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Note.—Published in October, 1933, *Proceedings*.

[1] Asst. Prof. of Civ. Eng., Univ. of Washington, Seattle, Wash.

[2] *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1209.

theoretical equations have been deduced. The standard design equations for concrete structures do not account for stresses in the steel and the concrete induced by the shrinkage of the latter, although experience and observations indicate that this neglected factor is of prime importance.

Concrete shrinks as it dries. This action induces initial stresses, compressive in the steel and tensile in the concrete. Experimental evidence indicates that these induced stresses, the magnitudes of which are not generally recognized, are very high.

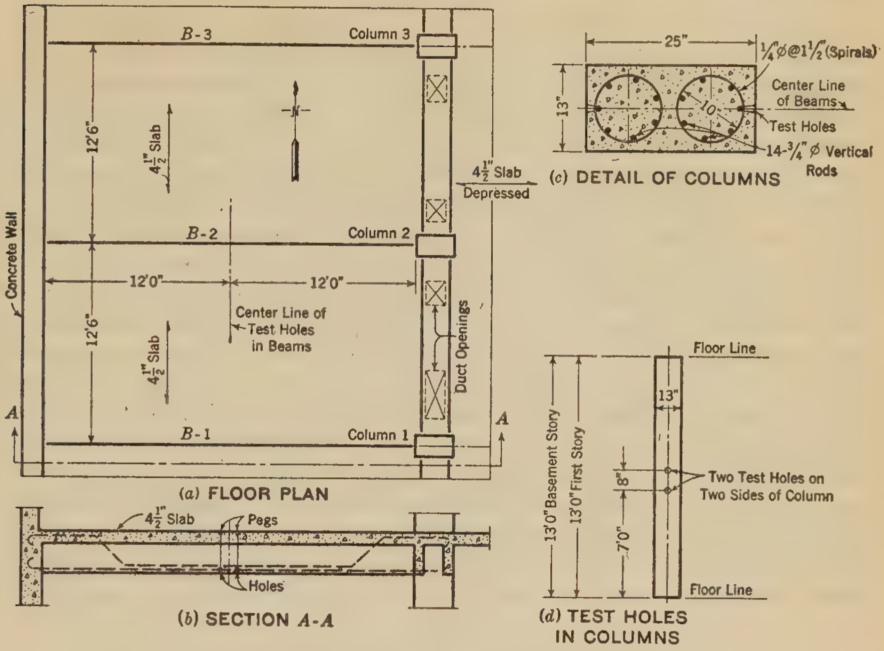
Shrinkage effects have been studied on a laboratory scale. Some readings on actual buildings have been taken, but to the writer's knowledge when this problem was begun, no investigations had been undertaken to measure the deformations of reinforcement in ordinary structural members in the early stages of construction, A review of the findings on the subject of shrinkage in concrete structures shows that a study of the effect of shrinkage on the deformation of steel during and after construction is a needed contribution to engineering knowledge.

Results of measurements—during and after construction—of hard-grade steel used in the building of Biology Hall at the University of Washington, Seattle, Wash., are presented herein. This is a five-story, reinforced concrete building of a beam-and-slab type. Observations were made on two faces of three columns of 1:1:2 concrete, three columns of $1:2\frac{1}{4}:3\frac{3}{4}$ concrete, and on each face of six beams of $1:2\frac{1}{4}:3\frac{3}{4}$ concrete. Readings were taken for a period of about two years, beginning on February 11, 1930.

The purpose of this work may be summarized as follows: (1) To observe the effect of shrinkage and flow of the concrete on the deformation of steel reinforcement; (2) to note whether the concrete and the steel act together during the early period of setting of the concrete; (3) to note the actual stresses in the reinforcement; (4) to observe the magnitude of the initial stresses; (5) to compare the observed stresses with the calculated theoretical stresses due to actual loads, and with the allowed stresses; and (6) to note the variation in the deformations over a long period of time. Fig. 1 shows the typical construction details in the building.

Instruments Used and Discussion of Readings

An 8-in. Berry strain-gauge with long legs was used, giving readings to $\frac{1}{40000}$ in. directly, and closer by interpolation. The temperature of the surrounding atmosphere was taken, but not of the specimens themselves. This introduces an error when the temperature difference between any two readings is large and the time interval small. The temperature of the specimen might be considered as lagging behind the readings. It was noted that different temperatures gave different zero readings on the reference temperature bar provided with the instrument; for example, for a temperature of 40°F the bar reading was about 221, whereas for a temperature of 80°F the bar reading was about 228. This condition is explained by noting that the materials used for the strain-gauge frame and the reference temperature bar have different coefficients of expansion. A comparison of the coefficients of expansion of the reinforcement steel and of the reference temperature bar, showed the two to be practically equal. Thus, by taking the bar readings with the actual readings of the specimen, the deformation due to temperature changes could be accounted for; for example, the deformation in a bar of one of the columns is obtained as follows:


B-3
Column 3
B-2
Column 2
B-1
Column 1
12'6"
12'6"
12'0"
12'0"
$4\frac{1}{2}$" Slab
$4\frac{1}{2}$" Slab
N
Concrete Wall
Center Line of Test Holes in Beams
Duct Openings
$4\frac{1}{2}$" Slab Depressed
A
A
$4\frac{1}{2}$" Slab
Pegs
Holes
25"
$\frac{1}{4}$" ø @ $1\frac{1}{2}$" (Spirals)
13"
10"
Center Line of Beams
Test Holes
14-$\frac{3}{4}$" ø Vertical Rods
Floor Line
13"
13'0" Basement Story
13'0" First Story
8"
7'0"
Two Test Holes on Two Sides of Column
Floor Line

(a) FLOOR PLAN

(b) SECTION A-A

(c) DETAIL OF COLUMNS

(d) TEST HOLES IN COLUMNS

FIG. 1.—TYPICAL DETAILS OF BUILDING TESTED.

	February 11, 1930 (no load; temperature 45° F)	February 13, 1930 (temperature, 40° F)
Actual reading of specimen..........	262.5	256.5
Reference bar reading...............	222.0	221.0
Difference	40.5	35.5

The shortening in this case, in 8 in., is found to be $\frac{40.5 - 35.5}{5\,000} = 0.001$ in.

PROCEDURE OF TEST

Steel test specimens were taken at random from a stock-pile. Holes were drilled at the proper places and initial readings taken before the specimens were placed in the forms. The next readings were taken as soon as possible after the concrete had been poured, generally from one to two days later. Readings were then taken daily for the first seven days, weekly for the first month, and monthly for the first six months. Additional readings were taken when shoring was removed. In a few cases, because of inaccessibility, the readings were not recorded. Every deformation reading is the average of two values, the instrument being turned end for end to eliminate errors. No effort was made to regulate the construction to suit the needs peculiar to the test.

It was difficult to obtain exactly the same conditions for the similar members tested. Two columns, in the basement story, were poured in one day while the third column was poured two days later. This was also true for the first-story columns. Under the circumstances, readings for the same age of similar members tested, were considered together. Thus, the readings of two columns having the same age were studied with the readings of the third column of the same age, although the latter was poured two days later.

A greater difference in age exists between the two sets of beams. On the first floor, two beams were poured in one day and the third beam a day later. All the second-floor beams were poured in one day, but nine days after the first-floor beams. Practically, these six beams are similar in percentage of end restraint, and in magnitude and distribution of load; they are equal in span and cross-section. Readings for the same age of all six beams were studied together, although the writer realizes that some discrepancy resulted from such a combination. On the whole, this discrepancy detracts little from the accuracy of the complete study of the problem because of the many other uncertain and unknown factors that enter into an investigation of this kind.

All concrete was ready mixed and delivered to the job in trucks. The length of haul is estimated at about $\frac{1}{2}$ mile. Graded gravel was used for the coarse aggregate. Tests of standard 6 by 12-in. cylinders, in accordance with the Building Code of the City of Seattle, showed the ultimate unit stresses at the age of 28 days to be as indicated in Table 1. No concrete was poured during freezing or near-freezing weather.

TABLE 1.—TESTS OF STANDARD CONCRETE CYLINDERS.

Location	Mix	Ultimate stress, in pounds per square inch	Number of cylinders tested
Basement columns	1:1:2	5 000	3
First-story columns	1:2¼:3¾	3 000	3
First-floor beams and slabs	1:2¼:3¾	3 750	4
Second-floor beams and slabs	1:2¼:3¾	3 750	4

NOTATION

The following symbols, adopted in this paper, are presented for the guidance of discussers:

a = actually computed.

c = concrete.

e = estimated.

f = unit stress, in pounds per square inch; subscripts: s = steel; c = concrete; u = ultimate strength; t = theoretical; r = shrinkage; a = actually computed; o = observed; superiors: f' = tensile stress; f'' = compressive stress.

n = ratio of the moduli of elasticity, steel to concrete; $n = \frac{E_s}{E_c}$.
o = observed.
p = ratio of areas, steel to concrete; $p = \frac{A_s}{A_c}$.
r = contraction.
s = steel.
t = theoretical.
u = ultimate strength.
A = area; subscripts: s = steel; c = concrete.
D = deformation in 8 in.
E = modulus of elasticity; subscripts: s = steel; c = concrete.
P = load, in pounds; subscripts: e = estimated; d = design.
W = weight of beam and slab, in pounds per foot.
ϵ = coefficient of contraction.

GENERAL

All stresses reported in this paper may be considered due to dead loads because, even after the building was finished and opened to occupancy, the loads superimposed during the period of observation were negligible. The following equation gives the general expression for unit stress, in pounds per square inch, due to deformation:

$$f = \frac{D\,E}{5\,000 \times 8}$$

in which, D is the gauge reading or deformation in 8 in. For steel the modulus of elasticity was found to be close to 30 000 000 lb per sq. in., and, for concrete, it will be assumed to be 2 000 000 lb per sq in. at 28 days. (In this paper no attempt was made to note the variation in the modulus of elasticity of concrete over a period of time.) For steel, the unit stress, in pounds per square inch, is:

$$f_s = \frac{30\,000\,000}{5\,000 \times 8} D = 750\,D \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

To compare the observed effects with the theoretical or calculated, care was exercised in observing the actual load on the member considered. For example, the load on a column with shoring in place is unknown; but without the shoring, the load can be computed accurately. A log of the construction was kept, and at any stage the load on any member could be estimated.

The shrinking process continues for a long time if the concrete is allowed to dry. A failure does not retard the action of the contraction. However, on account of a failure a member may cease to act as a unit. Tension failures in the concrete separate the member into smaller units, each unit contracting but not necessarily with a cumulative effect between given points. D. A. Abrams, M. Am. Soc. C. E., found that appreciably higher stresses were obtained by using 2-in. gauge lines as against 8-in. gauge lines. This is to be expected as there are fewer failures in the shorter gauge length.

From the theoretical equation for compressive stress in the steel due to shrinkage (based on the assumption that a member of constant cross-section contracts toward the center of the member) the maximum stress occurs at the center of the member.[3] The test points in the columns are located very nearly at that point, so that the observed stresses may be considered as the maximum compressive stresses in the steel.

Although concrete is an excellent building material, it is unsatisfactory from the standpoint of the theorist. That its modulus of elasticity is not constant has been proved conclusively. Strictly speaking, concrete is not an elastic material and, therefore, it can have no modulus of elasticity. However, a ratio of stress to strain can be obtained at any desired point on the stress-strain curve. Although much work has been done, it is difficult to arrive at the ratio of the moduli of steel to concrete for a given case, with a fair degree of accuracy. Constant values of n will be used in this paper for the purpose of comparison.

Disregarding the effects induced upon the steel and the concrete by shrinkage, stresses in plain reinforced columns are quite easily calculated. This is not the case for helically reinforced columns. Elastic longitudinal contraction in a column produces lateral expansion; and, conversely, elastic lateral expansion will cause longitudinal contraction. (This is the phenomenon noted by Poisson.) In a plain longitudinal reinforced concrete column, under an imposed load, the lateral expansion occurs without much restriction, as contrasted with the helically reinforced columns in which there is considerable restriction. This amounts to hindering longitudinal contraction reactively; and, since the stress is a function of the deformation, any manner of preventing elastic longitudinal contraction will strengthen the column. This fact is generally appreciated.[4] The stresses allowed by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete for helical columns compensate for the use of the helical reinforcement, but these are not the theoretical stresses. For comparative use, let the theoretical steel stress, f''_{st}, due to a load, P, be obtained, in pounds per square inch, from the equation:

$$f''_{st} = \frac{P\,n}{A_c + (n-1)\,p\,A_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which, A_c is the area of concrete; and $p = \dfrac{A_s}{A_c}$.

Basement Story Columns (Concrete Mix, 1:1:2)

Experimental Results.—The experimental readings are recorded by means of graphs. Curves in every case are drawn as the mean of the average readings, and do not necessarily pass through the average points. At best, these curves give the general trend of the variation in the deformations, and extreme accuracy in the individual readings can not be expected. In an investigation of this kind, there are too many non-controllable variables that affect the experimental results. However, the writer believes that the graphs do help one to appreciate the action taking place in the members under the average construction conditions.

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[3] *Bulletin No. 126,* Univ. of Illinois Eng. Experiment Station, p. 21.

[4] "Stresses in Helically Reinforced Concrete Columns," by A. F. Zesiger, M. Am. Soc. C. E., and E. J. Affeldt, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 379.

It is seen from Fig. 2 that the unit stresses for the first five days showed a definite increase; but on the sixth day a sudden decrease occurred. At this time the beams and slab of the floor above were poured, and the increase in load (if it did get to the columns) should have indicated an increase in the deformations. However, a definite decrease was observed. Apparently, minor failures or re-adjustments allowed the steel to return, partly, to its unstressed length. Some readers may question the occurrence of the sudden drop in the curves, and conclusive evidence can not be given to defend it. It is merely a personal interpretation of the experimental readings.

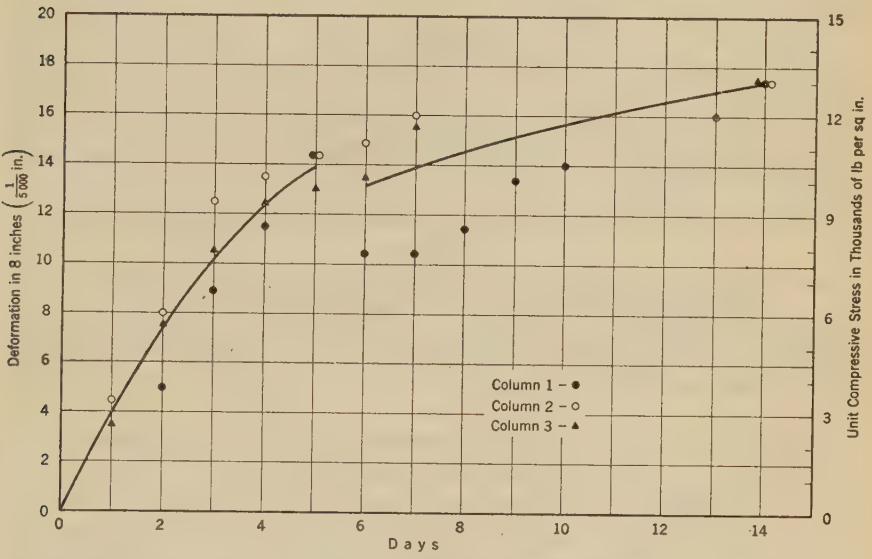

FIG. 2.—DEFORMATIONS AND UNIT COMPRESSION STRESSES OF STEEL IN WEST FACES OF BASEMENT STORY COLUMNS

Similar curves were plotted of the readings on the east faces of the basement columns. A close scrutiny of the two curves showed them to be almost identical for the first five days and differing from that time, until at fourteen days the east-face stresses were less than the west-face stresses, as they should be. The eccentricity of loading causes this difference. The curves for the concrete columns of lower strength bring out this detail much better.

Fig. 3 shows the average readings and the mean curve for a period of observation of approximately 2 years and 4 months. There is a gradual, almost constant, rise from 200 days to the end of the curve. The horizontal line, Curve 2, Fig. 3 ($f''_{st} = 12\ 500$ lb per sq in.) is governed by the formula,

$$f''_s = [300 + (0.1 + 4p)\ f''_{cu}]\ (n) \ldots\ldots\ldots\ldots (3)$$

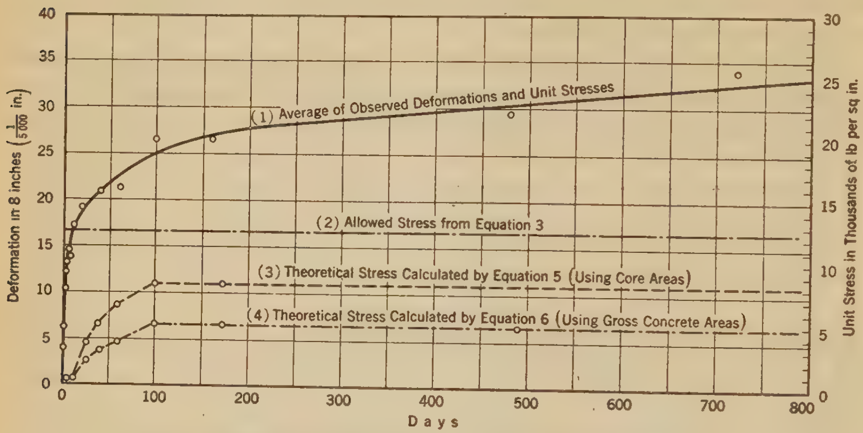

FIG. 3.—AVERAGE DEFORMATIONS AND UNIT COMPRESSIVE STRESSES IN EAST AND WEST FACES OF BASEMENT COLUMNS (CONCRETE MIX, 1 : 1 : 2).

in which, f''_s is the allowed compressive stress in steel, and f''_{cu} is the ultimate compressive strength of concrete. The value of n is taken as 12.

TABLE 2.—AVERAGE STRESSES INDUCED IN CONCRETE BY SHRINKAGE AND ACTUAL LOADS; CONCRETE MIX, 1:1:2.

(Tension is Positive and Compression Negative)

Age, in days (1)	Estimated load, in pounds (2)	Observed steel stress, f''_{so}, in pounds per square inch (3)	Load on steel, $A_s f''_{so}$, in pounds (4)	Load on concrete, in pounds (5)	Concrete stress, in pounds per square inch (6)
2	2 000	5 500	34 100	+32 100	+101
3	2 000	7 500	46 400	+44 400	+139
	(21 000)			(+44 600)	(+140)
7	7 000	10 600	65 600	58 600	+184
	(44 600)			(+34 000)	(+107)
14	7 000	12 700	78 600	+71 600	+224
27	67 000	14 600	90 400	+23 400	+ 73
40	90 000	15 800	97 800	+ 7 800	+ 24
61	120 000	17 100	106 000	—14 000	— 44
173	155 000	20 000	124 000	—31 000	— 97
300	160 000	21 500	133 000	—27 000	— 85
480	160 000	22 700	140 500	—19 500	— 61
600	160 000	23 500	145 000	—15 000	— 47
700	160 000	24 300	150 000	—10 000	— 31
850	160 000	25 500	157 600	— 3 400	— 10.7

A summary of the average stresses induced in the concrete by shrinkage and actual loads is shown in Table 2. In Column (2) the values in parentheses are the total estimated loads, which in all probability did not affect the deformation readings because shoring was in place up to 28 days.

The remaining values in Column (2) are the probable effective loads. The unit stress in the concrete is a computed value obtained from:

$$P_e = A_s f_{so} \pm A_c f_c \qquad (4)$$

in which, P_e is the estimated load; A_s, area of steel; f_{so}, average observed stress in steel; A_c, area of concrete; and f_c, unit stress in concrete. Every quantity in Equation (4) is known, except the concrete stress. The gross area minus the steel area was used for the effective, concrete cross-section; thus, $A_c = 13 \times 25 - 6.18 = 318.8$ sq in. Although the stresses are obtained by computation, nevertheless, they are the true mean values. There is some doubt about the actual loads during the first two weeks and, consequently, some uncertainty about the true induced stresses.

By plotting the values in Column (6), Table 2, against age (Column (1)), a clearer concept is afforded of the variation in the concrete stress. For the first 50 days there is tension in the concrete, followed by a reversal, until at 200 days a maximum compressive value is reached. From that time, the stress decreases under a constant load, and the burden is gradually transferred from the concrete to the steel by the shrinkage of the concrete. The average concrete stress at 850 days is 10 lb per sq in.

Calculated Theoretical Stresses.—The calculated theoretical unit stress due to actual loads in the steel for reinforced concrete columns, disregarding the effect of the helical reinforcement, is determined by substituting in Equation (2): $A_c = 2 \times 78.54 = 157.1$; $n = 12$ (assumed constant); and, $p = 0.0392$. Thus,

$$f''_{st} = \frac{P}{18.75} \qquad (5)$$

Equation (5) is obtained using the core areas only. Subsequent analysis shows that it is more reasonable to use the gross concrete area, namely, 13 by 25 in. = 325 sq in. Then, $p = 0.019$, and,

$$f''_{st} = \frac{P}{32.8} \qquad (6)$$

Equations (5) and (6) are represented graphically in Fig. 3. To obtain the apparent shrinkage effect on the steel, subtract the theoretical from the observed stress. Thus, the stress caused by shrinkage at 800 days is either 25 000 − 8 500 = 16 500 lb per sq in., or 25 000 − 4 900 = 20 100 lb per sq in., the latter figure undoubtedly approaching the actual condition obtaining in the column.

For concrete of this strength, recognized authorities recommend values of n from 6 to 10, which would give smaller theoretical unit stresses than $n = 12$, and would result in a greater difference between the observed and the calculated theoretical unit stresses.

There is every reason to believe that the concrete outside the helical reinforcement produces its share of shrinkage effects. Curve 4, in Fig. 3, is the one to use when comparing the observed stresses with the calculated theoretical stresses. Thus, at 850 days, the observed steel stress is five times greater than the theoretical stress.

Allowed Unit Stresses.—For the helically reinforced concrete columns observed in this investigation the allowed unit stresses as specified by the Joint Committee[5] are determined by Equation (3). Thus, if $p = 0.0392$; $f_{cu} = 2\ 900$ lb per sq in. at 28 days (recommended by the Joint Committee); $E_c = 2\ 500\ 000$ lb per sq in.; $n = 12$; (by Equation (3)) $f''_c = 1\ 045$ lb per sq in.; and f''_s = compressive stress in steel = $(n)\ (f_c) = (12)\ (1\ 045) = 12\ 550$ lb per sq in.

Furthermore,[5] the design load, P_d, on a column is computed by,

$$P_d = f''_c\ (A_c - A_s) + f''_s\ A_s \ldots\ldots\ldots\ldots\ldots\ldots(7)$$

or, $P_d = 1\ 045\ (157.1 - 6.18) + 12\ 550 \times 6.18 = 235\ 200$ lb. By comparison, the estimated dead load plus live load equals 230 000 lb.

There is no correlation between the observed and the allowed steel stresses. The load on the columns at 850 days is about 70% of the design load; nevertheless, the observed, is twice the allowed, stress. Were the full design load imposed on the columns the ratio probably would be greater than 2. With an increase in the load, the concrete would be stressed more, but not to the extent of 1 045 lb per sq in. (See discussion on "First-Story Columns.")

From the foregoing remarks, it must not be concluded that the steel is unsafe when stressed in compression beyond 12 550 lb per sq in. A better working stress to use would be 20 000 lb per sq in. The steel in a reinforced concrete column if kept from buckling, should be good for a stress of 80 000 lb per sq in., and more. However, when it is stressed beyond the elastic limit, excessive deformations will result with practically no increase in the load. It is then that the concrete takes its share of the load. The allowed stress is not an indicator of the actual stresses obtaining in the columns.

FIRST-STORY COLUMNS (CONCRETE MIX, 1 : 2¼ : 3¾)

Experimental Results.—At the end of the fifth day, after pouring, a re-adjustment occurred in the columns composed of the richer concrete. An inspection of Fig. 4 reveals that a similar action had taken place for these columns one day later. Corresponding curves plotted from readings on the west faces show the same action. This re-adjustment may have been caused by one, or by a combination, of the following: (1) Failure of concrete in tension; (2) failure in bond at the end of the rods; (3) failure in shear between the inner core and the outer hollow cylinder containing the reinforcing steel; (4) flow of the concrete around the deformed rods; and, (5) flow of the inner core of concrete. Whether or not other re-adjustments had occurred, it is difficult to say because readings were taken at too great intervals of time. However, it seems logical to expect large flow effects in the early stage of construction and tension failures after the concrete has taken a set.

All beams under observation were designed to have no end restraint. Curves corresponding to Fig. 4, for the west faces of the columns, show the

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[5] *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1204, Equation (43).

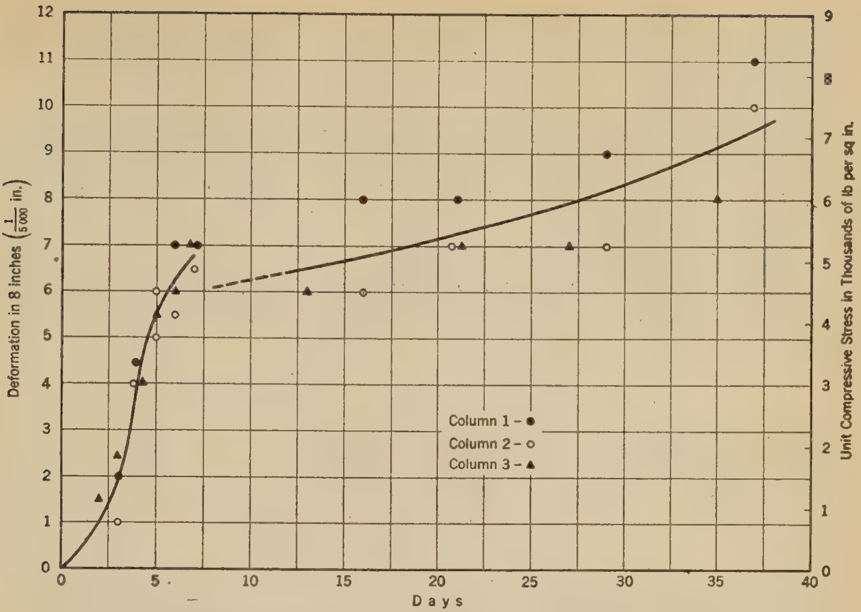

FIG. 4.—DEFORMATIONS AND UNIT COMPRESSIVE STRESSES OF STEEL IN EAST FACES OF FIRST-STORY COLUMNS.

same characteristic re-adjustment. A comparison of the stresses, however, at 28 days, shows that those on the west faces are 1 200 lb per sq in. more than on the east faces. This difference can be attributed to the eccentric loading on the columns, rather than to the continuity between the beam and

TABLE 3.—AVERAGE STRESSES INDUCED IN CONCRETE BY SHRINKAGE AND ACTUAL LOADS; CONCRETE MIX, 1:2½:3¾

(Tension is Positive and Compression Negative)

Age, in days	Estimated load, in pounds	Observed steel stress, f''_{so}, in pounds per square inch	Load on steel, $A_s f''_{so}$ in pounds	Load on concrete (Column (2) — Column (4)), in pounds	CONCRETE STRESS, IN POUNDS PER SQUARE INCH	
(1)	(2)	(3)	(4)	(5)	$A_c = 318.8$ (6)	$A_c = 157.1$ (7)
3...........	2 300	1 500	9 300	+ 7 000	+22	+ 45
4...........	7 000	2 600	16 100	+ 9 100	+ 29	+ 59
	(21 500)			(—5 100)	(—16)	(—32)
7...........	7 000	4 300	26 600	+19 600	+ 62	+126
	(44 000)			(—8 100)	(—26)	(—53)
14...........	10 000	5 800	35 900	+25 900	+ 81	+164
27...........	66 000	7 000	43 300	—22 700	— 71	—144
36...........	74 000	7 400	46 000	—28 000	— 88	—178
74...........	100 000	9 700	60 000	—40 000	— 126	—255
160...........	128 000	12 300	76 000	—52 000	—163	—330
200...........	136 000	12 900	80 000	—56 000	—176	—357
300...........	136 000	14 500	90 000	—46 000	—145	—294
400...........	136 000	15 700	97 000	—39 000	—122	—250
480...........	136 000	16 600	103 000	—33 000	—104	—211
600...........	136 000	17 700	110 000	—26 000	— 82	—166
700...........	136 000	18 500	115 000	—21 000	— 66	—134
850...........	136 000	19 600	121 000	—15 000	— 47	— 95

the column. Perfect continuity between the beam and the column would tend to diminish the difference in the stresses. A deflection diagram will indicate this roughly.

A summary of the true mean concrete stresses in the columns is given in Table 3. It must be noted that the period of induced tension in these columns is smaller in duration and magnitude than for the columns of 1:1:2 concrete. As in Table 2, the values in parentheses in Column (2), Table 3, are the total estimated loads.

Calculated Theoretical Stresses.—Let A_c = 157.1 sq in. (core areas); $n = 15$ (assumed constant); and $p = 0.0392$. Then, the theoretical unit stress in the steel due to load (disregarding the strengthening effect of the helix) is:

$$f''_{st} = \frac{P}{16.2} \qquad \text{.........................(8)}$$

If the gross concrete area is used, $p = 0.019$, and:

$$f''_{st} = \frac{P}{27.4} \qquad \text{.........................(9)}$$

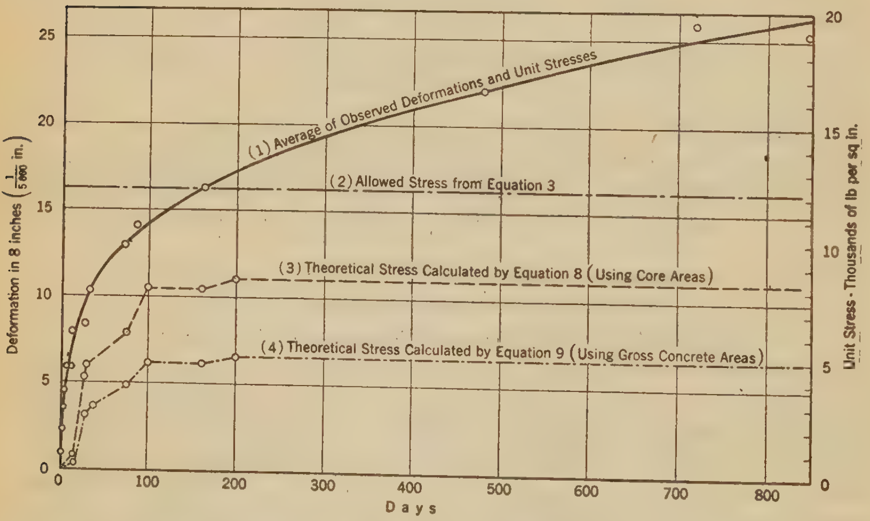

FIG. 5.—AVERAGE DEFORMATIONS AND UNIT COMPRESSIVE STRESS IN EAST AND WEST FACES OF FIRST-STORY COLUMNS. (CONCRETE MIX, 1 : 2¼ : 3¾).

In Fig. 5, Curves 3 and 4 show the theoretical stresses. Probably Curve 4 depicts the actual variation in the theoretical stresses better than Curve 3. It is seen that from 200 to 850 days, under practically constant load, the apparent shrinkage influence increases. This action can be attributed to the shrinkage of the concrete.

Shrinkage influence on the steel stress is measured by subtracting the theoretical stress from the observed. (See Fig. 5.) Thus, at 850 days the apparent shrinkage influence is 19 500 − 8 400 = 11 100 lb per sq in., or 19 500 − 5 000 = 14 500 lb per sq in., which is considerably less than the 16 500 and 20 100 lb per sq in., respectively, for the columns of rich concrete mixes.

The only reason for using the 1:1:2 concrete in the basement columns was to utilize its greater strength. The Joint Committee's equations[6] for the steel stress for both mixes, in this case, give almost the same result, but the observed steel stress at 850 days for the rich mix is 6 000 lb per sq in. more than for the leaner mix. The allowed concrete stresses are 1 045 and 814 lb per sq in. for the 1 : 1 : 2 and 1 : $2\frac{1}{4}$: $3\frac{3}{4}$ mixes, respectively. An examination of values in Column (6) of Table 2, and Columns (6) and (7) of Table 3 shows that at no time do the actual stresses approach these values. Furthermore, the rich concrete, except for the tensile load imposed upon it, carries a smaller part of the actual load than the leaner mix. Obviously, the rich mix defeats the purpose for which it was intended.

Allowed Unit Stresses.—Let $p = 0.0392$, $f_{cu} = 2\ 000$ lb per sq in. (recommended by the Joint Committee); and, $n = 15$ (assumed constant). Then, for helically reinforced concrete columns the allowed unit stress in the concrete (by Equation (3)) is $f''_c = 814$ lb. per sq in. For steel, the stress is: $f''_s = nf''_c = 12\ 200$ lb per sq in.

As in the case of the basement columns, the design load is computed by Equation (7), so that $P_d = 814\ (157.1 - 6.18) + 12\ 200 \times 6.18 = 198\ 500$ lb.

There is no correlation between the observed and the allowed steel stresses. At 850 days, the imposed load is approximately 70% of the design load. The probable unit steel stress under the full design load at 850 days, using the shrinkage effect of 14 500 lb per sq in., would be:

$$f''_s = f''_r + \frac{5\,000}{70\%} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

or, $f_s = 14\,500 + 7\,150 = 21\,650$ lb per sq in.

The probable unit concrete stress equals $f''_c = 50 + \dfrac{7\,150 - 5\,000}{15}$, or 229 lb per sq in. Apparently, therefore, the concrete strength would be utilized with an increase in the load, but the steel would show a greater over-stress.

Shrinkage.—The aim in the following computations is to compare the observed shrinkage effects with the theoretical. The theoretical maximum compressive steel stress due to shrinkage is:

$$f''_{sr} = \frac{3\ E_s\ \epsilon}{2\,(pn + 1)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and in the concrete:[7]

$$f'_{cr} = f''_{sr}\ (p) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which, f''_{sr} = unit compressive stress in steel due to contraction of concrete; and ϵ is the coefficient of contraction.

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[6] *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1204 *et seq.*

[7] *Bulletin No. 126*, Univ. of Illinois Eng. Experiment Station, p. 21.

From his experiments with limestone concrete, F. R. McMillan, M. Am. Soc. C. E., deduces that the average coefficient of contraction for plain air-cured 1:2:4 concrete at 30 days is close to 0.00025 in. per in.; at 60 days, it is 0.00050 in. per in.; and, at 90 days, it is 0.00055 in. per in. For all practical purposes the latter is the maximum coefficient.[8] According to tests performed by R. E. Davis,[9] M. Am. Soc. C. E., 1:2:3 gravel concrete at the age of three months, gives about twice as much contraction as limestone concrete, other factors being constant.

An inspection of Fig. 6 shows that the deformations at the tops of the beams (where contraction is almost unrestricted), caused by shrinkage and by dead load, more nearly approach Mr. McMillan's figures. These will be used in the following computations. At 36 days, ϵ is approximately 0.025 per cent. Using the net area of the concrete as $325 - 6.18 = 318.8$ sq in., $p = 0.019$, and $n = 15$: f''_{sr} (by Equation (11)) $= 8\,750$ lb per sq in. (compression); and, f'_c (by Equation (12)) $= 167$ lb per sq in. (tension). At 74 days, $\epsilon = 0.05\%$; $f''_{sr} = 17\,500$ lb per sq in.; and $f'_{cr} = 334$ lb per sq in. Other values are computed for different ages of concrete, and for $p = 0.019$, and $p = 0.0392$.

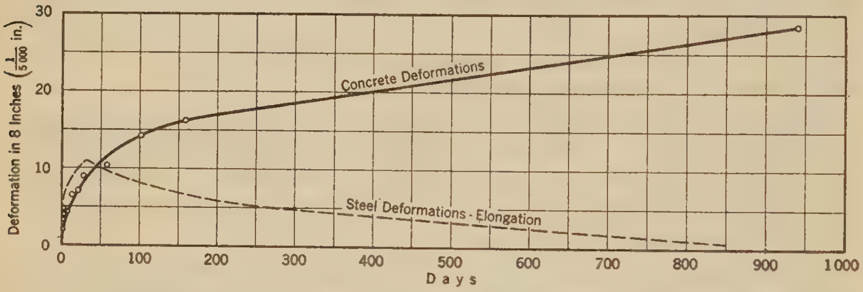

FIG. 6.—AVERAGE DEFORMATIONS OF CONCRETE AT TOPS OF FLOOR-BEAMS AND OF STEEL IN THE BOTTOMS OF FLOOR-BEAMS (CONCRETE MIX, 1 : 2¼ : 3¾).

The result of these computations is recorded in Columns (9) and (10) Table 4. A discussion of the results in this table is desirable. To obtain values for the unit stress in the concrete to compare with those in Columns (6) and (7), Table 3, the following procedure was used. Subtracting the theoretical steel stress (Column (4)), from the observed steel stress (Column (3)), gave the apparent stress due to shrinkage (Column (6)). According to theory, the tensile stress in the concrete induced by shrinkage (Column (7)) is the product of Columns (6) and (13). The actual stress in the concrete (Column (8)) is the algebraic sum of the theoretical and the induced stresses. This procedure is repeated for different ages and for two values of p. Using the theoretical stresses induced by shrinkage and the theoretical stresses due to loads, the values in Column (11) are obtained. Comparing the magnitude and character of the concrete stresses with those shown in Columns (6) and (7), Table 3, for the corresponding ages, it is noted that the values in

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[8] *Transactions*, Am. Soc. C. E., Vol. LXXX (December, 1916), p. 1740.

[9] *Journal*, Am. Concrete Inst., February, 1930, p. 419.

Column (8) for $p = 0.019$ and $p = 0.0392$ check favorably, the values for $p = 0.019$ giving the closer agreement. That the theoretical shrinkage stresses added to the theoretical stresses in the concrete due to load, do not agree with the values in Column (6) of Table 3, might have been anticipated since the "slips" between the steel and the concrete in the early period of construction were neglected in Column (9); but it is surprising to find such close correlation between Column (6), Table 3, and the upper half of Column (8), Table 4, despite the use of the uncertain value of n which appears only in one (Table 4) of the two groups of stresses compared.

TABLE 4.—COMPUTATIONS FOR TENSILE STRENGTH OF $1:2\frac{1}{4}:3\frac{3}{4}$ CONCRETE AND COMPRESSIVE STRESS IN STEEL DUE TO CONTRACTION OF CONCRETE.

(Tension is Positive, and Compression Negative)

Age, in days (1)	Estimated load, in pounds (2)	Observed steel stress, in pounds per square inch (3)	Theoretical compressive steel stress due to load, in pounds per square inch (4)	Theoretical compressive concrete stress due to load, in pounds per square inch (5)	Apparent steel stress due to shrinkage, in pounds per square inch (6)	Apparent concrete stress due to shrinkage, in pounds per square inch (7)	Actual concrete stress, in pounds per square inch (8)	Theoretical compressive steel stress, due to shrinkage, in pounds per square inch (9)	Theoretical tensile concrete stress, due to shrinkage, in pounds per square inch (10)	Actual concrete stress, in pounds per square inch (11)	Coefficient of contraction assumed (12)	p (13)
FROM OBSERVED READINGS								THEORETICAL				
36	74 000	−7 400	−2 700	−180	−4 700	+ 90	− 90	−8 750	+167	− 13	0.00025	0.019
74	100 000	−9 700	−3 750	−245	−6 050	+115	−130	−17 500	+334	+ 89	0.00050	0.019
160	128 000	−12 300	−4 680	−312	−7 620	+145	−167	−19 250	+366	+ 54	0.00055	0.019
850	136 000	−19 600	−4 970	−330	−14 630	+288	− 50	−19 250	+366	+ 36	0.00055	0.019
36	74 000	−7 400	−4 570	−305	−2 830	+110	−195	−7 070	+277	− 28	0.00025	0.0392
74	100 000	−9 700	−6 170	−412	−3 530	+138	−274	−14 140	+554	+142	0.00050	0.0392
160	128 000	−12 300	−7 900	−527	−4 400	+173	−354	−15 600	+612	+ 85	0.00055	0.0392
850	136 000	−19 600	−8 400	−560	−11 200	+440	−120	−15 600	+612	+ 52	0.00055	0.0392

EXPERIMENTAL RESULTS, T-BEAMS (CONCRETE MIX, $1:2\frac{1}{4}:3\frac{3}{4}$)

Steel in Beams.—A high tensile stress was developed in the first three days (under no load), dropping off considerably on the fourth day. (See Fig. 7.) The drop is larger than for the columns of the same mix. The difference may be attributed to the large size of rods used, which fail much easier in bond than the smaller rods for the same effect per square inch of cross-section.

From 7 to 30 days, the increase in the stress is quite rapid and fluctuating, giving indication of possible re-adjustments. Although shoring was removed during this time, no increase in deformations was noticed. In the period from 30 to 100 days, a large decrease was observed under constant load. (See Fig. 8.) The apparent shrinkage effect curve has the same general shape as the column curve for the same mix, showing that variation in stress is due to shrinkage.

The curve in Fig. 8 shows the variation of the steel stress for a period of 2 years and 4 months. Under a constant load, the steel is being gradually contracted. The average tensile stress at 850 days was obtained from twelve readings, some of which indicated compressive stresses. It may be expected that in the future compressive stresses in the steel may result, unless a re-adjustment occurs caused by a tension failure in the concrete.

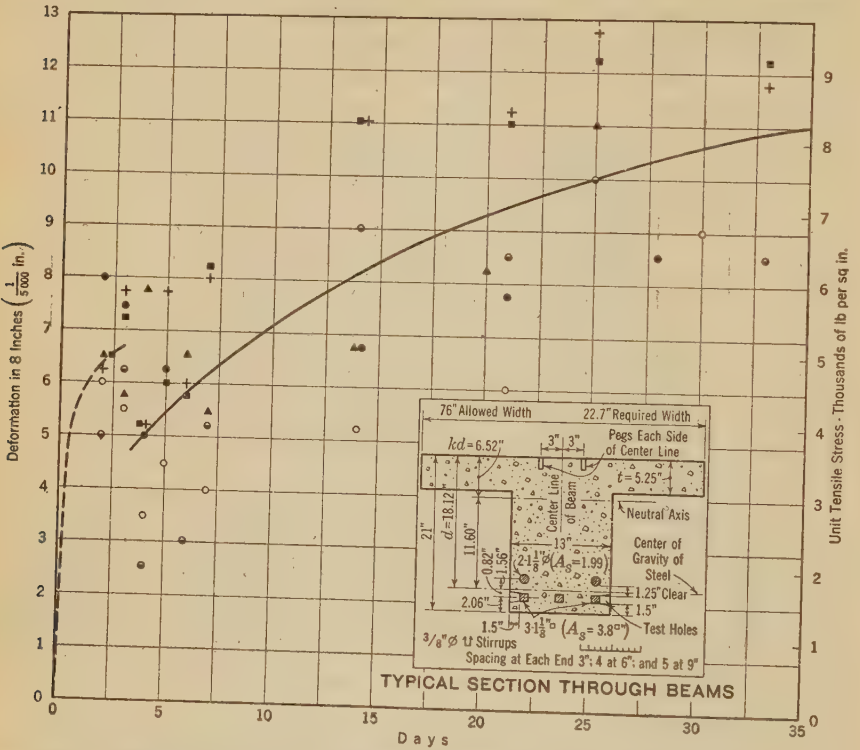

FIG. 7.—ELONGATIONS AND UNIT TENSILE STRESSES IN THE REINFORCEMENT OF THE MAIN BEAMS OF THE FIRST AND SECOND FLOORS. (CONCRETE MIX, 1 : 2¼ : 3¾).

To obtain an approximate measure of the effect of shrinkage on the beam steel at 850 days, it seems reasonable to use:

$$\left(\begin{array}{c}\text{Shrinkage}\\\text{stress}\end{array}\right) = \left(\begin{array}{c}\text{Calculated unit}\\\text{stress}\end{array}\right) - \left(\begin{array}{c}\text{Observed unit}\\\text{stress at 850 days}\end{array}\right)$$

or, 9 360 lb per sq in. (compression) $\doteq$ 9 860 — 500.

Concrete in Beams.—Fig. 6 shows the average of the concrete deformations at the tops of the beams. The points on the curve before 160 days represent an average of twelve readings obtained from six beams, and after 160 days an average of six readings obtained from three beams. Shrinkage of free concrete was not studied in this paper; therefore, it is not possible to obtain numerical values for the actual stresses in the concrete.

Calculated Theoretical Stresses.—The following considerations entered into the computation of stresses in the beams: (*a*) All beams under observation had no theoretical end restraint; (*b*) bending moment due to uniformly distributed load $= \frac{wl^2}{8}$ ft-lb; (*c*) dead load = weight of beam + weight of slab + $\frac{3}{4}$-in. finish; (*d*) weight of beam = 205 lb per ft of beam; (*e*) weight of slab = 787 lb per ft of beam; (*f*) total weight (Item (*d*) + Item (*e*), w, = 992 lb per ft of beam; and, (*g*) bending moment = 71 500 ft-lb.

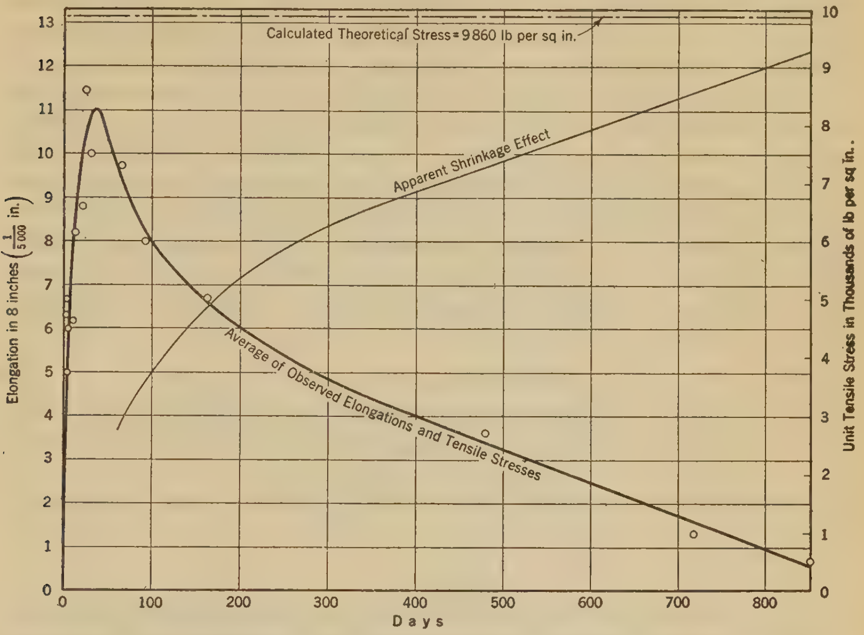

FIG. 8.—AVERAGE ELONGATIONS AND UNIT TENSILE STRESSES IN REINFORCEMENT OF MAIN BEAMS OF FIRST AND SECOND FLOORS.

Furthermore, the Seattle Building Code allows a **T** of: b = width of beam + thickness of slab = 13 + 2 × 6 × 5.25 = 76 in.; f_s = 20 000 lb per sq in.; f_c = 750 lb per sq in.; n = 15 (assumed constant); and $\frac{t}{d} = \frac{5.25}{18.12} = 0.29$. For the foregoing stresses, $k = 0.36$; $p = 0.0065$;

$$K'' = 115^{10}; j = \left(1 - \frac{t}{d}\right) \frac{3\,k - 2\,\frac{t}{d}}{6\,k - 3\,\frac{t}{d}} = 0.888; \text{ and } jd = 0.888 \times 18.12 = 16.1 \text{ in.}$$

Therefore, $f'_s = \frac{M}{A_s\,jd} = 9\ 200$ lb per sq in. (calculated stress);

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[10] "Structural Members and Connections," by Hool and Kinne, p. 469.

f'_s (extreme) $= 9\,200 \dfrac{(11.60 + 0.82)}{11.60} = 9\,860$ lb per sq in.; $Kbd^2 = M = 71\,500 \times 12$; and $b = 22.7$ in., which is the required width.

The extreme fiber stress was obtained by the ordinary design method. The approximate value of k equal to three-eighths would give a result just as accurate considering the uncertainties; and neither one probably represents the true conditions in the beam. However, for comparison, the stress of 9 860 lb per sq in. will be used. It will be noted in Fig. 8 that the observed stress never reached the calculated theoretical stress. At 850 days the observed stress is only 500 lb per sq in., much lower than is generally suspected. With the steel stress as low as it is, the question arises as to how the beam is able to carry the load. The answer is to be found in combining the stresses produced by the load and the shrinkage deformations. This phase of the problem needs further study, but it appears that it is not entirely a matter of theory. Tests must be made to determine the actual behavior of beams with different percentages of reinforcement.

Allowed Unit Stresses

The allowed unit stress for hard grade steel reinforcement is 20 000 lb per sq in. If the specified live load of 60 lb per sq ft were imposed, the probable total stress at 850 days would be: $500 + \dfrac{60\ (12)\ (9\,860)}{992} = 8\,660$ lb per sq in., which is safe.

Conclusions

The experimental data presented in this paper justify the following conclusions, assuming that all concrete areas in the columns, unless otherwise noted, are taken as the gross net areas; that is, 318.8 sq in.:

1.—Concrete and steel do not act together during the first seven days after pouring, whereas the usual assumption is that they act together at all times.

2.—The initial steel stresses are high. Initial stresses are understood to be those which exist in the member before the load is applied. In construction similar to this, the load can be carried by the member only when the shoring is removed, in this case at the end of 28 days:

(*a*) For the rich-mix concrete column at 28 days there is an initial compressive steel stress of 12 500 lb per sq in., which is slightly more than that recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete for the maximum design stress in a column of this type and for this percentage of longitudinal reinforcement.

(*b*) The leaner-mix concrete column at 28 days shows an initial compressive steel stress of 4 500 lb per sq in., about one-third the allowed stress. Thus, the shrinkage and flow of the concrete impose an extra load on the steel for which no allowance is made in the design.

3.—The actual unit compressive stresses in the columns of rich-mix concrete at 2 years and 4 months was 25 000 lb per sq in. for the steel and 10 lb per sq in. for the concrete, whereas the allowed stresses are 12 550 lb per sq in. and 1 045 per sq in., respectively. Accordingly, there is not even a reasonable correlation between the actual and the allowed unit stresses, although it should be noted that the imposed load is 70% of the design load. If the full design load were applied to the column, the observed stresses probably would both be greater, leaving the concrete safe and the steel greatly overloaded.

4.—The actual unit compressive stresses in the column of leaner-mix concrete at 2 years and 4 months was 19 500 lb per sq in. in the steel and 50 lb per sq in. in the concrete, the allowed stresses being 12 700 lb per sq in. and 814 lb per sq in., respectively.

5.—The actual unit tensile stress in the beam steel at 2 years and 4 months was 500 lb per sq in., whereas the allowed stress is 20 000 lb per sq in. The calculated theoretical stress in this case due to dead load alone is 9 860 lb per sq in.

6.—The influence of the shrinkage and flow of the concrete on the steel at 2 years and 4 months shows that there was:

(*a*) Nearly 20 000 lb per sq in. compressive stress developed in the rich-mix concrete column;
(*b*) Approximately 14 000 lb per sq in. compressive stress developed in the lean-mix concrete column; and,
(*c*) About 9 360 lb per sq in. compressive stress in the beam.

7.—The effect of framing a beam on one side of a column produced flexural stresses in the column. The magnitude of these stresses at 28 days was 600 lb per sq in. compression in the steel nearest the beam and 600 lb per sq in. tension in the opposite steel.

8.—No additional deformation was observed in the members when shoring was removed. It is possible that the load was gradually transferred to the members by the drying of the form lumber and shoring.

9.—Shrinkage should be considered as a property of concrete, and design equations should be altered to include the effect of shrinkage of the concrete both on the steel and on the concrete.

10.—The empirical stresses, recommended by the Joint Committee for allowed stresses in columns, give no indication of the actual stresses prevailing in the column at this early stage of loading.

It would be poor engineering to deduce general equations for beams and columns from the results recorded in this paper, because of the comparatively small number of members tested. However, these observations have shown that it is questionable to use rich concrete for columns without making due allowance for the increased compressive stress induced in the steel. In general, the shrinkage of the concrete greatly increases the stress in the steel and decreases that in the concrete. This action for the 1 : 1 : 2 concrete columns gives a steel stress at 850 days 50% higher and a concrete stress 80% lower than the corresponding stresses in the 1 : 2¼ : 3¾ columns. When loaded to failure helically reinforced columns of rich concrete show greater strength than columns of leaner mixes because of the redistribution of stresses after the elastic limit of the steel is reached.

The compressive steel stress in the columns is higher than is generally suspected. It is evident that variation in the percentages of steel used in the columns will give different results. A correlated study of columns with different percentages of steel subjected to increasing load increments approximating actual conditions, is desirable. These tests should simulate the average construction conditions, particularly in the manner and medium by which the load is imposed on the column. It is important to put the load on the concrete of the column as in actual construction. Loads thus applied tend to "strip" the concrete from the steel. If steel plates are used at the ends of the column and there is contact with the longitudinal reinforcement, a condition is created which does not approach the actual, and the "stripping" factor is neglected.

From the foregoing data and remarks, it may be concluded that the average column design favors the concrete but not the steel. Based on the allowed steel stress, the designs are unsafe. The steel in the 1 : 1 : 2 columns is heavily stressed. It has been proved by tests that the shrinkage phenomenon goes on for many years; eventually, the yield point of the steel is reached resulting in large deformations under a constant load. When this occurs the concrete will take a larger share of the imposed load, a fact which may explain why some reinforced concrete columns do not fail when seemingly they should.

The beam design is quite safe for the steel and the concrete. When it is realized that shrinkage does affect the steel stress, a comprehensive study of beams should make it possible to use this factor in the design.

Future investigations should record readings at shorter intervals of time, especially in the early stage of construction. Shrinkage of unstressed, plain concrete should be recorded for comparison and study of the experimental results. This would be useful in obtaining the concrete stresses due to load at the top of the beams, and would aid in developing a design equation involving shrinkage of the concrete.

In conclusion, it may be stated that although the imposed loads on the members tested did not reach the design loads, the condition is representative of many cases met in practice. Furthermore, engineers are inclined to study the strength of reinforced concrete structures under breaking loads, and to slight their behavior under actual conditions. It is hoped that, in the future, tests on the beams discussed in this paper, will be made with the full design loads.

ACKNOWLEDGMENTS

The writer is greatly indebted to Charles C. May, Assoc. M. Am. Soc. C. E., for his kindness in permitting the use of the Biology Building and for his valuable suggestions; to The Hendrikson-Alstrom Construction Company, of Seattle, for its friendly co-operation during the period of testing; to Messrs. A. M. Winslow and A. L. Miller, and to M. O. Sylliaasen and C. W. Harris, Members, Am. Soc. C. E., and I. L. Collier, Assoc. M. Am. Soc. C. E., for their helpful comments, particularly to Professor Miller for his critical perusal of the details of the findings.

DISCUSSION

William Russell Davis,[11] M. Am. Soc. C. E. (by letter).—By emphasizing that the shrinkage of concrete over a long time should be kept constantly in mind by designers of structures in which concrete is used, this paper is made a most valuable contribution.

The writer has observed the results of long-time shrinkage in the increasing deflection of concrete beams that have reinforcement, for the most part, in the bottom only. The effect of such shrinkage has been the distortion of door frames, followed by the binding of doors, and the cracking of partitions. It is evident that the shortening of the unreinforced top, by shrinkage, must be greater than the shortening of the reinforced bottom, and that this relative change in length must be accompanied by an increased deflection which does not impair strength, but is a cause of anxiety to the owner of a building when doors bind and partitions crack. These results can be avoided by using adequate steel beams under partitions, or by using, in the top of reinforced beams that carry partitions, an area of reinforcement approximately equal to that used in the bottom. In fact, since the greater the percentage of steel the less the change in length from shrinkage, it may be predicted that, by using more steel in the top than in the bottom, the deflection will decrease as shrinkage takes place.

The author suggests that further experiments may yield data about shrinkage, which will lead to material changes in the formulas now used for the design of both beams and columns. The data cited in this paper appear to justify such an expectation in the case of columns, but the case for beams is very different. Commenting on Fig. 8, Professor Sergev states that some readings indicated compression in the bottom reinforcement of beams at 850 days and under "Calculated Theoretical Stresses," he states that "all beams under observation had no theoretical end restraint."

The bending moment at the center of the beam must be counteracted by a resisting moment that can only exist if there is tension at the bottom. If there is no tension in the steel, the concrete must be in substantial tension despite the fact that it should not be considered as dependable in tension. In the case of the beams in question, it cannot be shown that the concrete was not stressed almost to the point of rupture at 850 days. These beams were observed under dead load only. The application of the live load might have caused rupture and produced a stress in the steel closely approximating the value given by the formula of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. That formula was never intended, and does not profess, to show the stress intensities that actually prevail prior to rupture of the concrete. The formula is predicated on the acceptance of the fact that concrete is not reliable in tension and that rupture is likely to occur. Just as long as concrete is not reliable in tension, such tests as those described in this paper, despite an excellent showing for the tensile quality of the concrete, cannot justify any modification of the beam formulas now in general use.

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[11] Cons. Engr., Albany, N. Y.

J. R. WORCESTER,[12] M. AM. SOC. C. E. (by letter).—A striking example is presented in this paper, of the effect of the shrinkage of concrete during the process of hardening and for some time thereafter. Although the fact that shrinkage, under normal building conditions, is to be expected, and although the likelihood of stresses in the reinforcement at variance with the usual design assumptions has been generally admitted, there has been little definite information, until recent years, as to the extent of such variance, the speed with which it may develop, or the time during which it may continue to increase.

As the author agrees, the results give insufficient data for the deduction of general rules for design, not only because of the comparatively small number of members tested, but particularly because the tests were not carried far enough to develop even the stresses under the design load, not to mention the stresses at the ultimate load. It is true, as suggested, that it seems inconsistent to base designs upon conditions of load that are not expected ever to be reached, but the impossibility of securing a desired factor of safety, without knowing what happens after a working load is passed, is apparent in the case of the beams described in the paper. It is evident that with the dead load alone the tensile strength of the concrete was not exceeded. No light is shed on what would have developed with a slight increase in load.

Professor Sergev's assumption that the beams were freely supported at the ends does not seem to be justified by the details shown in Fig. 1. It would appear from this diagram that the connection over the double columns was strong enough to develop the flexural strength of the column, and to have restrained the girder almost completely. The consideration of this restraint would have modified some of the calculations involving the beams. The maximum positive moment under the dead load would have been about 40 000 ft-lb instead of 71 500. Under this moment the tensile fiber stress in the concrete would have been only about 355 lb per sq in., a stress that might well have been resisted without fracture. It is obvious, however, that with a comparatively slight increase of load the concrete would have cracked, and the tension in the reinforcement would have been increased.

Perhaps the most surprising result obtained in this investigation was that "the rich mix [used in the lower section of columns] defeats the purpose for which it was intended." Apparently, this is due to the shrinkage being greater with the richer mix. The greater stress in the steel from this cause would probably continue as the load increased until the stress in the steel reached the yield point, but at that point there would be a re-adjustment, and the greater compressive strength of the richer concrete would have manifested itself.

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[12] Cons. Engr., Boston, Mass.

There will be general agreement with the author's Conclusion 9, that "shrinkage should be considered as a property of concrete, and design equations should be altered to include the effect of shrinkage of the concrete both on the steel and on the concrete." In the matter of beams, where the effect of shrinkage is to increase the danger of cracks in the concrete, it is not apparent how the usual assumption that the concrete carries no tensile stress can be superseded.

F. E. Richart,[13] M. Am. Soc. C. E. (by letter).—The paper by Professor Sergev performs a useful service in again demonstrating to engineers that actual stresses and design working stresses in reinforced concrete columns are rarely the same. It is hardly correct to say that a "comparison of these [Joint Committee working] stresses with the observed stresses indicates the departure of the actual from the anticipated," because no one should expect the Joint Committee formula to indicate the actual stress at just any stage of development of flow and shrinkage. On the other hand, and contrary to the author's statement, the Joint Committee formula does allow for the occurrence of stresses in columns due to a definite and reasonably large amount of shrinkage and plastic flow; also, because of a varying factor of safety, it discourages the use of small steel percentages which would result in undue steel stresses due to time yield effects. The Joint Committee formula was the logical sequel to the classic paper by McMillan in 1921,[14] in which the importance of shrinkage and flow was comprehensively treated. Tests of the effect of flow and shrinkage formed a large part of the column investigation, carried on by the American Concrete Institute from 1930 to date (1933), and more than 170 columns of varied design have been under sustained loading and observation for 1 year to 3 years.

It would have been desirable had the author distinguished between the effects of shrinkage and flow. This could have been done by taking readings on "dummy" or unloaded specimens to determine the shrinkage. As it is, in Tables 2 and 3, the stresses are listed as due to "shrinkage and actual loads." Similarly, the modulus of elasticity of the concrete might well have been determined, instead of assuming it to be 2 000 000 lb per sq in. With the cylinder test strengths of exactly 5 000, 3 000, and 3 750 lb per sq in., as given in Table 1, the value of *E* should doubtless be much higher, although, as the author states, the effective value appears to be lower from the test data. This would naturally result from plastic flow.

The location of gauge lines on bars before they are placed in the form, and the reversal of the strain-gauge end for end during readings, are operations from which great accuracy can hardly be expected. Differences in manner of holding the gauge and slight bending of the bar in placing it in the form will introduce large errors. These methods would hardly be expected to support Conclusion 7, for example, which states that framing a beam into one side of a column produced flexural steel stresses of 600 lb per sq in.

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[13] Research Prof. of Eng. Materials, Univ. of Illinois, Urbana, Ill.

[14] "A Study of Column Test Data," by F. R. McMillan, M. Am. Soc. C. E., *Proceedings*, Am. Concrete Inst., 1921, p. 150.

in compression and tension, on the respective sides of the column. This stress corresponds to four-fifths of a division of the gauge dial and is within the normal error of observation.

The break in the time-deformation curves of Figs. 2 and 4 of the paper seems hardly necessary. In Fig. 2, the points for two of the columns show no marked break, but great importance is evidently attached to the variation in readings for the third one. In Fig. 4, the break in the curve is introduced because of two observations which are about one dial division off of the regular trend of the curve. The writer can see no great significance in this part of the time-deformation curve and cannot see therein the basis for the conclusion that "concrete and steel do not act together during the first seven days after pouring, whereas the usual assumption is that they act together at all times." Certainly bond is developed at early ages. Abrams[15] has showed that the bond strength at 2, 4, and 7 days for concrete of this general grade is just as great a proportion of the compressive strength as it is at 28 or 60 days.

The statement that, in the case of columns, "the rich mix defeats the purpose for which it was intended," would seem to be a rather unfortunate generalization. While the columns of rich concrete did show steel stresses of 25 000 lb per sq in. after 2 years and 4 months as compared to 19 500 lb per sq in. for the leaner concrete, the difference is not alarming. Much greater differences in steel stresses might be expected from a variation in percentage of longitudinal steel than that due to a variation in concrete mixture.

The conclusion that "based on the allowed steel stress, the designs are unsafe," may be questioned. In the first place, the factor of safety involved in the Joint Committee formula is rather large, especially in the region of small steel percentages. Furthermore, it has been well established that initial shrinkage and yield stresses in columns have no effect upon the ultimate strength when the column is loaded to failure. The outstanding feature of such columns is the undesirable amount of shortening that may be produced by time effects under working loads.

The writer agrees with the author that the effect of shrinkage, and plastic flow as well, should be recognized and provided for in design. Obviously, a design formula cannot contain terms involving variations in shrinkage, flow, modulus of elasticity, strength, and other properties which occur as the structure ages, but it must be devised to cover such combinations of these time and load effects as may produce unfavorable or dangerous conditions in the structure.

HOMER M. HADLEY,[16] ASSOC. M. AM. SOC. C. E. (by letter).—The results of a large amount of observational work, carried on for nearly two and one-half years, are summarized in this paper, which is one of those contributions to the store of engineering knowledge that slowly and gradually bring about a truer understanding of the behavior and ways of Nature. Professor Sergev reports reality from that greater laboratory where Nature works with full-sized models. Even if the exact conditions of testing cannot be carefully controlled, there is a compensating advantage in the elimination of errors of interpretation and judgment which may occur when reasoning from the behavior of small models to the expected behavior of larger ones.

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[15] "Tests of Bond Between Concrete and Steel," by D. A. Abrams, M. Am. Soc. C. E., *Bulletin 71*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[16] Regional Structural Engr., Portland Cement Assoc., Seattle, Wash.

The author's findings are in accord with those of the Committee on Reinforced Concrete Columns of the American Concrete Institute. Shrinkage in the concrete induces compressive stresses in the reinforcement which, depending upon the relative cross-sectional areas of the two materials, the various physical and chemical properties of the concrete, its age, etc., are of varying magnitude but, in general, are of high value when contrasted with the values commonly assumed in current design.

It is always something of a shock to find that theories one has accepted unquestioningly and has fully believed in, are not so. Some of Professor Sergev's terms and expressions indicate that he has not completely adapted himself to the change from the aforementioned Joint Committee's maximum allowable stresses to those he observed. Regardless of unconventional values, however, attention must not be diverted from the fact that the Joint Committee procedure and rules "work"— they produce structures that are staunch and safe — and, as indicated by the **T**-beams in the paper, may be excessively safe from the viewpoint of economy.

Undoubtedly, the reinforced concrete column must be considered in a new light, in view of these and similar observations in recent years. That in numerous cases shrinkage stresses in the concrete in combination with the column loads have stressed the reinforcement to the yield point cannot be questioned. Mere probability and the tens of thousands of concrete columns in existence assure the certainty of this; and yet the columns function successfully. If adequate lateral support and bracing are provided by the enclosing concrete, by spiral, or by numerous closely spaced binders or ties, why should not longitudinal reinforcement be stressed to the yield point when at that juncture the practically unstressed concrete is capable of assuming a great additional load?

Table 2 reports unit stresses in the 1:1:2 columns at 850 days as 25 500 lb per sq in. in the reinforcement and 10 lb per sq in. in the concrete; and the total column load as being 160 000 lb. Should this entire load be doubled by some means and should all the increase come upon the concrete, it would result in an additional stress of 492 lb per sq in., making a total concrete stress of 502 lb per sq in., with concrete which Table 1 shows to have a cylinder strength of 5 000 lb per sq in. There does not appear to be anything alarming in this prospect, particularly when the doubled column load is purely hypothetical and has no conceivable source or origin in the building.

Toward the end of the paper Professor Sergev states that "the average column design favors the concrete, but not the steel. Based on the allowed steel stress the designs are unsafe." Furthermore, he states that when, eventually, the yield point of the steel is reached and subsequently large deformations occur with any increase in load, "the concrete will take a larger share of the imposed load, a fact which may explain why some reinforced concrete columns do not fail when seemingly they should." The suggestions in this wording are not the happiest. The evidence submitted in no wise indicates lack of safety in the columns nor incipient failure. All that is shown is a very complete lack of concordance between assumed unit stresses and actual unit stresses. It is the assumptions which are shown to be weak and in need of strengthening—not the columns.

F. N. Menefee,[17] M. Am. Soc. C. E. (by letter).—Shrinkage cracks in concrete vary from the minute "crazing" cracks seen only by the aid of a magnifying glass (and which are as shallow as they are minute) to the large cracks one sees in floors, retaining walls, beams, girders, and columns where no provision has been made to prevent their forming and which may extend completely across the member. Since these latter cracks were preceded by tension in the member, in most cases induced by shrinkage, and since good concrete under favorable conditions may stand a tension of 400 lb per sq in. for a short time, it should be obvious that considerable concrete is in tension, varying from small insignificant amounts to amounts just below failure, after which the tension in the concrete at the point of failure is relieved.

Laboratory tests on mortars were probably first made by Considére, who found 1:3 plain mortars shrunk from 0.05% to 0.15% when hardened in air. Neat cements shrink more, as was to be expected, since the sand resists shrinkage and the compression induced by shrinkage, and the volume of water is greater. These values have been checked many times since, the actual values depending on the amount of cement, the quantity of water, and the nature of the matrix. In general, with all other conditions the same, rate of evaporation of the water determines the elapsed time before the maximum deformation is reached. In the test recorded in Fig. 9 the forms were of steel; hence the rate of shrinkage was relatively slow.

In a general way the outside limits of the compression in the steel due to shrinkage of the concrete may be arrived at by considering the tensile strength of the concrete, which seldom exceeds 400 lb per sq in. ultimate. In the columns investigated by the author the gross area of the concrete basement column was 325 sq in. If, due to shrinkage, the full value of 400 lb per sq in. was developed in the concrete, the total tension would amount to 130 000 lb, which must necessarily be resisted by compression in the steel. With 6.2 sq in. of steel, this amounts to 21 000 lb per sq in., which is approximately the value observed by the author on the basement columns at the end of 800 days, after the computed stress due to load is subtracted.

The foregoing illustration is possible even if some re-adjustment of stresses seems to have taken place at about the sixth day because, after the re-adjustment, shrinkage still continues for at least two or three years, although at a slower rate, particularly after the first year. The break in the average of the deformations in Figs. 2 and 4 at the age of 5 or 6 days may be due to re-adjustments or minor failures, as the author indicates, because work on the structure may have set up vibrations which caused the shrinking concrete to fail in tension. At this age ordinary 5 000-lb concrete would not stand much more than 250 lb per sq in. of steadily applied tension. According to Fig. 2 the unit compressive stress in the steel was 10 500 lb per sq in. (amounting to a total of 65 000 lb), which, in turn, would be the amount of tension in 325 sq in. of concrete, or 200 lb per sq in. This stress on 6-day concrete, with vibrations incident to pouring a floor over it, might easily cause tension failure in the concrete with a consequent relief of the compression in the steel.

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[17] Prof., Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

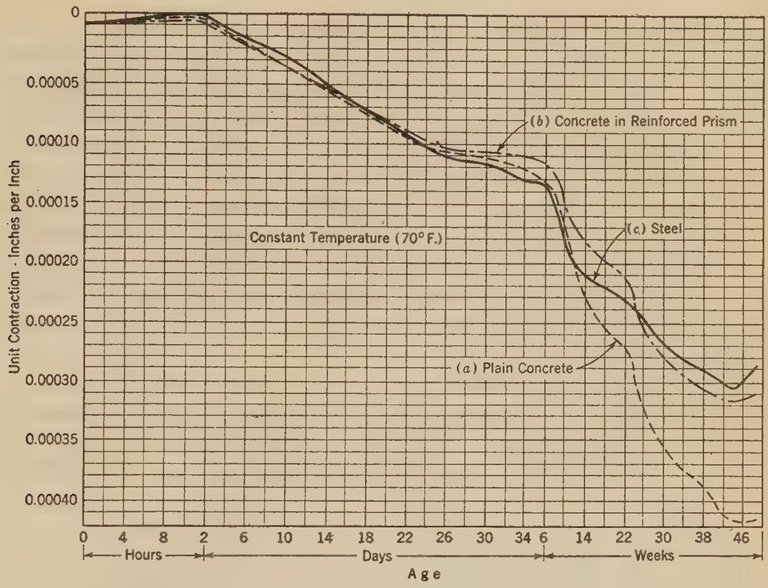

FIG. 9.—SHRINKAGE-TIME DIAGRAM FOR STEEL AND CONCRETE IN A REINFORCED CONCRETE PRISM AND FOR CONCRETE ALONE IN A NON-REINFORCED PRISM

From the results of his own tests, designed to determine when gripping of the concrete is complete, the writer believes that it is not likely that there was a failure of bond at six days, because the maximum bond stress was only 26.5 lb per sq in., using 0.000015 in. per in. as the shortening of plain concrete in the formula[18]:

$$P_y = \frac{uol}{4}\left[1 - \left(\frac{2\,y}{l}\right)^2\right] \dots\dots\dots\dots\dots\dots\dots (13)$$

in which, P_y = the total compressive stress in the bar = the total tensile stress in concrete, at any section, y, distant from the center; u = the maximum bond stress at the end of the bar; o = the circumference of the bar; and l = the length of the prism. This occurred at the upper and lower ends of the column, tapering off to zero at the center. If the concrete experienced a change in volume when in a plastic state, say, before initial set had taken place, the probabilities are that plastic slip or flow along the rod would occur, but the writer believes that some time between initial and final set, plastic slip ceases.

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[18] *Bulletin No. 126,* Eng. Experiment Station, Univ. of Illinois, Equation (1), p. 20.

The value of 0.000015 was taken from Curve (*a*) Fig. 9. It is the unit shortening of unreinforced concrete at six days. This value is not reliable for general use. While it is accurate for the cement in the particular prism, it is known that other prisms made by other cements gave higher results. One in particular, concerning which there was some doubt, ran ten times as high (which would have given 265 lb per sq in. of bond stress). This is about the value at which pull-out tests show incipient failure by slipping; but in the case at hand the steel is being pushed out of the concrete which, as far as bond resistance is concerned, is entirely different. A push-out test will show a greater resistance to bond stress than a pull-out test because, in the former case, the steel is expanding whereas, in the latter case, it is contracting.

The results of these tests seem to indicate complete grip at final set if not a little before. This was surprising since the concrete has so little tensile strength at that stage. The tests were run on prisms, 4 by 6 in. in cross-section by 29 in. long. Gravel concrete of 1:2:4 mixture was used; the maximum size was ½ in. The steel was placed in the center of the cross-section and varied from ⅜-in. round (deformed), ½-in. square (deformed), to ¾-in. plain and deformed round bars or, from 0.458 to 1.84 per cent. The bars were set in the concrete as shown in Fig. 10, with the ends turned up to make them accessible for measuring deformation. A constant length-measuring reference gauge was used, in which any change in length of one part, due to temperature, was offset by an equal change in the opposite direction by another part (see Fig. 11). A roller with an extension arm and mirror was placed between the brass bearing-block and the material to be measured. Readings were made by means of a telescope and a scale set at a distance in relation to the radius of the knife-edged roller, so that $\frac{1}{32}$ in. on the scale meant a change in length of the steel or concrete of 0.00001 in. The constancy of the length of the metal frame resting on the roller was determined by means of water baths in which the temperature could be controlled. Any change in length of the reinforcement steel or concrete produced a rotation of the mirror and a change in the scale reading as viewed through the telescope. Readings were begun 15 min after the water was placed on the con-

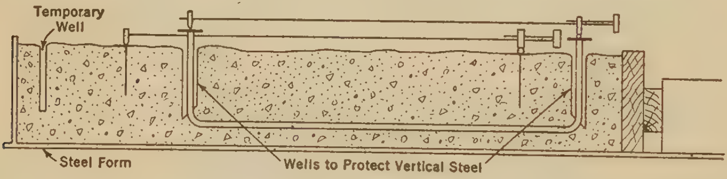

FIG. 10.—SET-UP FOR GAUGES

crete (which, previously, had been thoroughly dry-mixed) and were continued every few minutes throughout the first few hours. Then they were reduced in frequency as time went on until the end of seven days, and, finally, were taken once every two weeks.

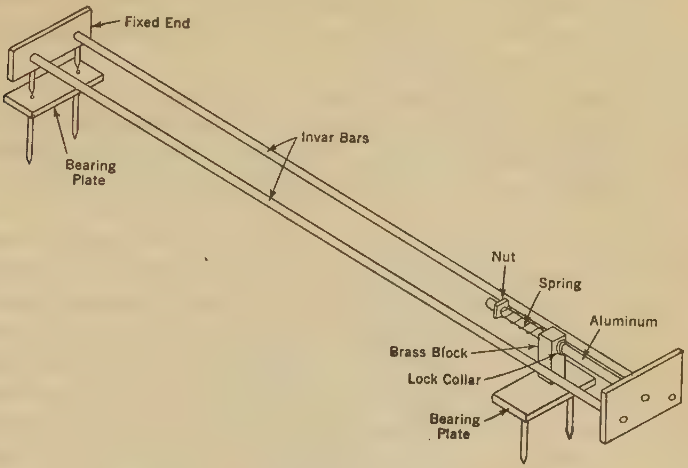

FIG. 11.—CONSTANT LENGTH REFERENCE GAUGE

In general, the results of this investigation indicated that by the end of the second hour the steel and the concrete were shrinking at the same rate. For some unexplained reason the steel seemed to shrink faster during the first two hours than the concrete. For some time this led the writer to believe that there was something wrong with the reference gauge or the measuring devices; however, with no change or interference on the part of the observers, the shrinkage curves of the concrete and steel generally ran very close to parallel after the end of the second hour.

Fig. 9 is a 48-week record of two prisms exactly alike, except that one (Curve (*b*)) is reinforced and the other (Curve (*a*)) is not. It will be noted that the plain concrete prism as well as the reinforced one increased in length for about a day, after which shrinkage began; the length passed from positive to negative on the third day, after which the shrinkage continued at about the same rate. Not all the prisms tested showed an increase in length in the early stages of their history. In some cases the shrinkage was noticed at the very beginning. For some reason which still remains unexplainable by the writer, the steel generally shortened at a greater rate during the first two hours, after which the shrinkage curves for steel and concrete were substantially parallel.

In other cases where the concrete increased in volume the reaction began just before, or at the time of, final set, and shrinkage set in. Of twelve prisms, all but two showed practical parallelism of the change-in-length curves for steel and the concrete after the second hour. It seems, however, that this might depend somewhat on the percentage of steel.

In submitting a discussion to this valuable paper the writer wishes to compliment the author on his technique and the thoroughness with which he has interpreted the data. Considerable laboratory work has been done, which rather indefinitely forecasted the results he has obtained, as well as some changes in column design specifications. In studying the columns in the field, the author has "clinched" many of the arguments relative to induced stresses due to shrinkage.

The writer dissents for the time being, at least, on what may be a minor point, namely the author's first conclusion. The experiments herein referred to, but which are not completed, seem to indicate that as far as bond is concerned the steel and concrete do act together after the time of final set.

Sergius I. Sergev,[19] Esq. (by letter).—Whenever the properties and the cross-section of a given member are required to describe completely the actions in, or on, the member, this denotes statical indeterminacy. The inclusion of the property of shrinkage in a reinforced concrete member leads to such a condition. Essentially, this means that before the effects of shrinkage are known the member itself must be known. This is undesirable, but is precisely the case in all indeterminate structures. Thus, shrinkage does not introduce an entirely new idea, but merely another point of observation from which reinforced concrete structures may be studied more thoroughly.

Mr. Davis' observations of excessive deflections in members are probably the result of shrinkage and plastic flow. In his discussion he is not definite as to the location of the concrete, when he remarks that "it cannot be shown that the concrete was not stressed almost to the point of rupture at 850 days." There is reason to believe that the concrete may not be ruptured at the top of the beams in the middle of the span, where the elastic deformation allows shrinkage to take place more easily than elsewhere in the cross-section. At the bottom of the beam the elastic deformation is in opposition to the shrinkage deformation; here, the concrete may, and probably does, eventually, fail in tension. However, there is much concrete in the average rectangular beam, or **T**-beam, which is effective in producing contraction in the steel, varying from maximum effect per unit area in the immediate vicinity of the steel to a minimum at the most distant fibers. Perhaps a better expression for "concrete is not reliable in tension" is "concrete has a low tensile strength."

The writer agrees with Mr. Worcester in the desirability of securing a factor of safety from tests for failure to destruction, but believes that theoretical equations should give exactly, or very closely, the actual conditions existing in the members. From the true facts approximations may be made.

Professor Menefee's discussion is enlightening in that he sums up roughly the underlying principle of the action of shrinkage. The shrinkage effect in the theoretical reinforced concrete equations is merely an elaboration of this thought. Regarding the first conclusion, the writer was aware of the bond intensity equation based on the assumption that the stress is maximum at the end, and minimum at the middle, of the embedded bars. Since the observation points were in the middle of the members, the first conclusion could not refer to the bond, it was merely a statement of fact.

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[19] Asst. Prof. of Civ. Eng., Univ. of Washington, Seattle, Wash.

In his suggestions, in his second and third paragraphs, Professor Richart is correct. The shortcomings he mentions were realized too late. Regarding the accuracy of the readings and of Conclusion 7, the following may be said: The writer's experience showed that, with proper manipulation, consistent readings might be obtained at a given time within the smallest division of the dial. Many readings were recorded as halves of the smallest dial division. One division represents a steel stress of 750 lb per sq in. Reversing the instrument was merely a precaution, and since all readings represented such a procedure, the writer does not understand how that detracts from their accuracy. Further, Conclusion 7 was based on the averages of two curves which, at 28 days, were 1 200 lb per sq in. apart. This represents 1.6 divisions on the dial and not 0.8 as Professor Richart states. The chief purpose of Conclusion 7 was to show that the stresses on the two sides of a column do not vary much from each other, thus indicating that the effect of eccentric loading due to the beam is negligible.

The breaks in the time-deformation curves of Figs. 2 and 4 may not be necessary, but since this drop occurred in all three sets of members under observation, although not at the same time, the writer thought it best to call attention to this action. Contrary to Professor Richart's statement, no special weight was given to any column, or member, and only the difficulty in drawing an average curve led to the suspicion of re-adjustment. In Fig. 2 the readings for Column 1, were taken daily, which was not the case for the other columns. This was due to lack of time for taking readings. Professor Richart should concede the possibility for such breaks unless numerous tests show the contrary to be true.

Mr. Hadley cites that under twice the load, namely, 320 000 lb, the unit concrete stress in the 1 : 1 : 2 columns at 850 days would be 502 lb per sq. in. This would probably be the case, but the steel would show a unit stress of about 31 000 lb per sq. in. This is close to the yield point of the steel. Usually, structures are designed for allowed stresses half-way between zero and the yield point. Reinforced concrete columns do not conform to this rule.

As Mr. Hadley observes, the statement, "the concrete will take a larger share of the imposed load, a fact which may explain why some reinforced concrete columns do not fail when seemingly they should," is misleading. The steel in such a case is stressed beyond the allowed stress, but not to the point of failure. In any practical column, it is possible to conceive of the steel failing in column action only. In helically reinforced columns this is not likely. This statement, therefore, should read somewhat as follows: "The concrete will take a larger share of the imposed load, a fact which may explain why some columns give good service with the steel greatly overstressed."

The writer appears to have misunderstood the intent of the recommendations of the Joint Committee on Reinforced Concrete Columns. Frankly, he has never attempted to obtain the chronological events leading up to these recommendations. He has only read and studied them. As far as he is aware, there is nothing to indicate that the equations for the columns do, or do not, describe approximately the actual conditions obtaining in them. Observations show that they indicate nothing definite, and although they have served a useful purpose in the past, they should be superseded by new equations which, in some measure, anticipate the actual conditions.

The discussers are to be commended for their unbiased and valuable comments, for the thought, and for the time consumed. The writer and undoubtedly others have profited much by their remarks.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1889

IRRIGATION HYDRAULICS[1]
FINAL REPORT OF SPECIAL COMMITTEE

To the Board of Direction,
American Society of Civil Engineers:

Gentlemen.—The Committee was appointed June 30, 1922. It was discontinued December 31, 1933, at the request of the Committee itself. The reasons for this request were:

(1) The Committee has been in existence for 11½ years and has contributed much to the advancement of the Engineering Profession during that period.

(2) The Committee has been very active throughout its existence while funds for its work were available. It has generally held two meetings per year and each member has had definite assignments of work.

(3) When funds were curtailed the work suffered to such an extent that it was thought the better policy would be for the Committee to suspend work altogether, resting on its record of accomplishments, rather than to continue a desultory existence. The Society's funds would thus be conserved rather than spent aimlessly.

The Board of Direction at its meeting of June 25, 1933, acquiesced in this view and discharged the Committee with a sincere vote of thanks and an allotment of funds to complete its work and close its affairs.

In addition to those signatory to this report the following have been members of the Committee: Messrs. Julian Hinds, Robert A. Monroe, A. L. Sonderegger, William Franklin Allison (deceased), Samuel Fortier (deceased), and Stuart H. Sims.

In the 11½ years of its existence, the Committee has held 17 meetings, at which the average attendance has been more than 80 per cent.

Plan of Procedure

At its first meeting a plan of procedure was outlined that has been closely followed. At that time the Committee planned:

1.—To select certain major subjects on which the Committee's activities would be concentrated.

2.—To prepare first a complete bibliography covering those subjects.

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[1] Presented at the Annual Meeting, New York, N. Y., January 17, 1934.

3.—To appoint as co-operating members engineers who have had special training and opportunities for study and research in each of the subjects selected.

4.—To stimulate the preparation of papers covering these subjects by those having special knowledge and competency in their preparation.

5.—To bring to light much valuable data and information in annual and special reports in the files of Federal and private agencies, that otherwise would be lost to the Engineering Profession.

6.—To stimulate original research by those having opportunities and to aid in securing funds therefor.

7.—To arrange for the installation of suitable apparatus in new structures to test their behavior after being placed in operation, such as piezometer pipes in siphon spillways, pressure pipes in dams, facilities for testing flow in lateral flow spillways, etc.

8.—To test fundamental theories by laboratory experiments enlisting the aid of facilities and operators in the hydraulic laboratories of engineering colleges and elsewhere.

9.—To co-operate with all agencies doing work of a similar nature, and to avoid duplication of efforts among groups of the Society.

Subjects Selected.—About twenty subjects were suggested and considered, of which the following were first selected by vote as being most worthy of study: (I) Evaporation from Reservoirs; (II) Canal Conversions; (III) Water Movement and Pressure Under Dams; (IV) Siphon Spillways; (V) The Silt Problem; (VI) Chutes and Drops; (VII) Scouring Below Dams; (VIII) Irrigation Deliveries; (IX) Lateral Flow Spillways; and (X) Permissible Canal Velocities. Subsequently, the following were added: (I-*A*) Evaporation from Soils; and (XI) Fishways and Fish Screens.

The Committee's work could not be restricted to irrigation alone, as nearly every subject listed is just as important in other lines of hydraulic engineering.

In addition to securing data on each of the subjects listed, the Committee prepared a complete bibliography in each subject. It has also prepared a glossary of terms and a set of standard symbols for writers in these subjects.

The tangible evidence of the Committee's activities is contained in the papers it has secured and presented to the Engineering Profession through publications of the Society and others. These are summarized briefly as follows:

Bibliography.—All important publications were searched for pertinent papers, resulting in a comprehensive bibliography. This bibliography (*Proceedings,* Am. Soc. C. E., March, 1925, Society Affairs, p. 147), gives the reference and a synopsis of the article.

Standard Symbols and Glossary of Terms Used in Hydraulics as Applied to Irrigation.—The Committee has labored about five years in the preparation of a list of standard symbols and a glossary of terms used in hydraulics as applied to irrigation. The preliminary work was published in *Proceedings,* Am. Soc. C. E., May, 1932, p. 729. The discussion which followed was voluminous. It was not published, but was transmitted to the Committee. These comments were taken seriously and most of the definitions were revised in conformity with the information and suggestions supplied. Since then, mimeographed copies of the revised glossary have been sent to about 200 hydraulicians with a request for additional suggestions. It is expected that the return from these requests will result in a final draft for publication as a manual, or in other suitable form, by the Society.

(I) and (I-A).—Evaporation from Reservoirs and Soils.—The Committee's work in these subjects was completed in 1932, and by agreement further consideration was relinquished to the Irrigation Division. One important objective the Committee had was to determine a set of standard coefficients by which records of evaporation from a large variety of pans could be converted to that from a large water surface. To this end a copper-lined reservoir 84 ft in diameter was constructed at Fort Collins, Colo., by the United States Bureau of Agricultural Engineering, in co-operation with the Colorado Experiment Station. Elaborate observations were made for a period of several years. The final paper—A Symposium on Evaporation from Water Surfaces—contains a summary of most of the existing evaporation data.

The papers secured on these subjects through the Committee's efforts are:

(1) "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 90 (1927), p. 266.

(2) "Experiments to Determine Rate of Evaporation from Saturated Soils and River-Bed Sands," by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 94 (1930), p. 961.

(3) "Water Supply from Rainfall on Valley Floors," by A. L. Sonderegger, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 94 (1930), p. 1242.

(4) "Evaporation as a Function of Insolation," by Burt Richardson, Esq., *Transactions,* Am. Soc. C. E., Vol. 95 (1931) p. 996.

(5) Evaporation from Water Surfaces: A Symposium: "Evaporation from Different Types of Pans," by Carl Rohwer, Assoc. M. Am. Soc. C. E.; "Evaporation from Reservoir Surfaces." by Robert Follansbee, M. Am. Soc. C. E.; and "Standard Equipment for Evaporation Stations," Final Report of Sub-Committee on Evaporation of the Special Committee on Irrigation Hydraulics, *Transactions,* Am. Soc. C. E., Vol. 99 (1934), p. 671.

(II).—Canal Conversions.—A large mass of data scattered through the files of the United States Bureau of Reclamation was assembled and supplemented by field measurements. Laboratory experiments have also been made. The assemblage of data appearing as a result of the Committee's efforts includes:

(1) "The Hydraulic Design of Flume and Siphon Transitions," by Julian Hinds, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 92 (1928), p. 1423.

(2) "Determining the Energy Loss in the Hydraulic Jump," by J. C. Stevens, M. Am. Soc. C. E., *Engineering News-Record,* July 22, 1926.

(3) "The Hydraulic Jump in Standard Conduits," by J. C. Stevens, M. Am. Soc. C. E., *Civil Engineering,* October, 1933, p. 565.

One of the perplexing problems before hydraulicians is energy losses in expanding conduits. The Committee has secured considerable data without, however, arriving at any definite conclusions. Among these data are the results of many experiments undertaken largely at the instance of the Committee, as follows:

(4) "Experiments on the Flow Through Flume Transitions": A Thesis, by Philip F. Thayer and J. Perry Yates, Juniors, Am. Soc. C. E., University of California, Berkeley, Calif., prepared as a result of laboratory experiments. A synopsis of this Thesis appears in *Proceedings,* Am. Soc. C. E., March, 1927, Society Affairs, p. 127.

(5) "Loss of Head in Expanding Conduits": A Thesis, by Messrs. Samuel Finlay and Jorge Altimirano, University of Santiago, Chile. This Thesis gives the results of about 830 experiments on flow through expanding flumes of varying characteristics. The basic data for these experiments were recomputed by J. C. Stevens, M. Am. Soc. C. E., and somewhat more consistent results obtained. This review is in the form of a paper in the Committee's files.

(6) "Experiments on Expanding Section," by Ralph Parshall, Assoc. M. Am. Soc. C. E., a paper prepared as a result of experiments at Bellvue Laboratory, Fort Collins, Colo.

(7) "Open Channel Expansion": A Thesis, by D. D. Curtis, Assoc. M. Am. Soc. C. E., prepared as a result of a series of experiments made in 1931 at the Hydraulic Laboratory of the University of Iowa, Iowa City, Iowa.

(III).—Water Movement and Pressure Under Dams. The Committee has been responsible in part for the systematic installation of pipes in dams both recently completed and now under construction for determining uplift pressures, not only at bed contacts with the foundation rock, but also in horizontal construction joints. Certain laboratory experiments have been made and others are in progress. Four important papers are available as a result of the Committee's work:

(1) "Upward Pressure Under Dams; Experiments by the United States Bureau of Reclamation," by Julian Hinds, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 93 (1929), p. 1527.

(2) "Water Uplift in Construction Joints of Concrete Dams": A Thesis, by the late John E. Skafte, Jun. Am. Soc. C. E., California Institute of Technology, Pasadena, Calif.

(3) "Uplift Pressure Measurements at Masonry Dams," by Ivan E. Houk, M. Am. Soc. C. E., *Civil Engineering,* September, 1932, p. 578.

Using the material gathered, D. C. Henny, M. Am. Soc. C. E., at the request of the Committee, prepared the following paper. It presents the theory of uplift in an entirely new light, and develops a rational method of treating this most important factor in the design of dams:

(4) "Stability of Straight Concrete Gravity Dams," by D. C. Henny, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 99 (1934), p. 1041.

(IV).—Siphon Spillways.—The Committee has induced the builders of several recently constructed siphons to install a series of piezometers so that their behavior may be tested. These include: Fuller Lake Siphon, Nevada Irrigation District, California; Dwinnel Reservoir Siphon, Montague Water Conservation District, California; Tiger Creek Conduit Siphon, Pacific Gas and Electric Company, Mokelumne River, California; and Leaburg Siphons, McKenzie River, City of Eugene, Oregon.

In addition, much laboratory experimental work has been done. The following papers contain the essential data secured by the Committee:

(1) A translation and recomputation of the losses given by E. Scimeni in his paper, "Sul Rendimento dei Sifoni Autolivellatari," *Electrotechnica,* May 5, 1928. (In Committee's files.)

(2) "Experiments on a Model Siphon": A Thesis, by Messrs. Brunner Gramatky and Kenneth Robinson, California Institute of Technology, Pasadena, Calif.

(3) "The Design and Operation of Siphon Spillways," a Thesis, by Jason Plowe, University of California, Berkeley, Calif.

(4) "Siphon-Spillway Models Tested against Prototypes," by Herbert H. Wheaton, Assoc. M. Am. Soc. C. E., *Engineering News-Record,* August 18, 1932.

(5) "On the Behavior of Siphons," by J. C. Stevens, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 99 (1934), p. 986.

(V).—The Silt Problem.—Many data have been gathered on the silt content of streams, the silting of reservoirs, and the control of silt in canals and reservoirs. The Committee's Final Report has been prepared by Mr. Stevens at the request of the Committee, the Silt Committee of the Power Division, and the Executive Committee of the Irrigation Division, as follows:

"The Silt Problem," by J. C. Stevens, M. Am. Soc. C. E. *Proceedings,* Am. Soc. C. E. (Publication pending).

In addition, the following have appeared in the Society's publications largely as a result of the Committee's efforts:

(1) "Report on the Silt Problem," by Fred D. Pyle and Franklin Thomas, Members, Am. Soc. C. E., with additions by the Committee, *Proceedings,* Am. Soc. C. E., March, 1925, Society Affairs, p. 141.

(2) "Silting of Reservoirs," with Bibliography, by Kirk Bryan, Esq., *Proceedings,* Am. Soc. C. E., March, 1927, Society Affairs, p. 129.

(3) "Sand Control Works at Fort Laramie Canal Intake," by Ivan E. Houk, M. Am. Soc. C. E., *Engineering News-Record,* June 14, 1928.

(4) "Sand Problems at Franklin Canal Intake, Rio Grande Project," by Ivan E. Houk, M. Am. Soc. C. E., *Western Construction News,* November 25, 1928.

(5) The status of the silting of Elephant Butte and McMillan Reservoirs is set forth by R. F. Walter, M. Am. Soc. C. E. in discussion of the paper,

"Silting of Lake Austin," by T. U. Taylor, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 93 (1929), p. 1719.

(VI).—Chutes and Drops.—An attempt was made to have field tests made on the hydraulic behavior of existing structures. Tentative promises of appropriations from the Division of Agricultural Engineering, U. S. Department of Agriculture and the U. S. Bureau of Reclamation were made, but owing to other demands these were never realized and the Committee postponed further consideration of this subject until some co-operative arrangements, with the necessary funds, could be secured. In its place the subject of "Fishways" was taken up.

(VII).—Scouring Below Dams.—The Committee's first endeavor was to secure a summary of existing information on this subject. The results appear as a Thesis on "The Scouring Effect of Water Below Dams," by I. Neudatchin (name since changed to Ivan M. Nelidov), University of California, Berkeley, Calif., a synopsis of which appears in *Proceedings,* Am. Soc. C. E., March, 1927, Society Affairs, p. 125.

Perhaps the most comprehensive collection of data on this subject in existence has been brought out through the Committee's efforts in the following paper and the discussions prompted by it:

(1) "Baffle-Pier Experiments on Models of Pit River Dams," by I. C. Steele and R. A. Monroe, Members, Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 93 (1929), p. 451.

(VIII).—Irrigation Deliveries.—The necessity for suitable devices for measuring irrigation water in any volume under all conditions of silting, aquatic plant growth, and where heads for weirs or other devices were not available, is apparent. With the Committee's aid and encouragement the Parshall Measuring Flume (formerly called the "Improved Venturi Flume") has been developed. It was in the making when the Committee was organized. Mr. V. M. Cone made the first Venturi flume and from his idea has been evolved the present Parshall measuring flume as a result of co-operative efforts between the U. S. Bureau of Agricultural Engineering and the Colorado Experiment Station, in which R. L. Parshall, Assoc. M. Am. Soc. C. E., was most active. There are Parshall flumes of many sizes now in service, measuring discharges from 1 to 3 000 sec-ft.

Another important phase of this subject is the quantity of irrigation water actually delivered to farm lands. The following papers have appeared at the request of the Committee:

(1) "The Improved Venturi Flume" (now called the Parshall Measuring Flume), by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 89 (1926), p. 841. The data in this paper with some additions and revisions are contained in *Bulletin 336,* Colorado Experiment Station, Fort Collins, Colo., under the same title and authorship.

(2) "Use of Water on Federal Irrigation Projects," by E. B. Debler, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 94 (1930), p. 1195.

(3) "Measuring Irrigation Deliveries in the Panjab," by E. S. Lindley, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1005.

(IX).—Lateral Flow Spillways.—Spillways of this character are becoming of increasing importance. Two important papers have been secured by the Committee as follows:

(1) "Side Channel Spillways; Hydraulic Theory, Economic Factors, and Experimental Determination of Losses," by Julian Hinds, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 881.

(2) "Side Spillways for Regulating Diversion Canals," by W. H. R. Nimmo, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 1561.

Mr. Hinds' analysis is predicated on the hypothesis that the energy of flow over the spillway crest is entirely lost at right angles thereto in the direction of outflow. In order to develop this phase of the problem the Committee secured the following papers:

(3) "Theoretical Energy Losses in Intersecting Pipes," by J. C. Stevens, M. Am. Soc. C. E., *Engineering News-Record*, June 4, 1925.

(4) "Experiments on the Losses in Intersecting Pipes": A Thesis, by Lawrence P. Sowles, Jun. Am. Soc. C. E., and M. Bernard McGowan, Jr., Assoc. M. Am. Soc. C. E., College of Civil Engineering, University of California, Berkeley, Calif.

(5) "A Study of Impact Effect:" A Thesis by Messrs. S. Gale Herrick and J. Frank Jorgensen, University of California, Berkeley, Calif.

(X).—Permissible Canal Velocities.—The Committee's first activity was to send out a questionnaire to irrigation managers, engineers, and others competent to speak with authority on this subject. The data thus secured, together with other data in hand, were presented in a final report on this subject, which appeared as:

(1) "Permissible Canal Velocities," by the late Samuel Fortier and Fred C. Scobey, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 940.

(XI).—Fishways and Fish Screens.—This subject is becoming increasingly important to the West, especially on those streams where migratory fish are caught commercially. The present status of existing knowledge on this subject is well presented in a paper recently secured by the Committee, as follows:

(1) "Problems of Fishway Construction," by Shirley Baker, M. Am. Soc. C. E., and U. B. Gilroy, Assoc. M. Am. Soc. C. E., *Civil Engineering*, December, 1933, p. 671.

In conclusion it should be remembered that the Committee has had no funds except for the traveling expenses of its members for their attendance at meetings and for clerical expenses of the Secretary's office. The expenses of field and laboratory work were paid entirely by other agencies that were induced to undertake work for the Committee.

The single task of having unearthed many valuable data on the several subjects under investigation, that would otherwise have remained buried in the files of the U. S. Bureau of Reclamation would itself have fully justified the existence of this Committee. The presentation of the data secured through papers under the names of individuals is believed to be a sound policy. It stimulates research and authorship, places credit where justly due, and elicits discussion that ofttimes is fully as valuable as the paper itself; but more important than all, it places in permanent form for reference, data that otherwise would ultimately be quite lost to the profession were the Committee's productions limited by its progress reports.

Respectfully submitted,
J. C. Stevens,
Secretary.

November 15, 1933.

Special Committee on Irrigation Hydraulics:

D. C. Henny, *Chairman,*
J. C. Stevens, *Secretary,*
B. A. Etcheverry,
J. L. Savage,
Fred C. Scobey,
Franklin Thomas,
R. L. Parshall,
Ivan E. Houk,
George W. Hawley,
I. C. Steele,
M. P. O'Brien.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 1890

TRENDS IN ENGINEERING AS A PROFESSION IN THE UNITED STATES OF AMERICA

ADDRESS AT THE ANNUAL CONVENTION, VANCOUVER, B. C., CANADA, JULY 11, 1934

BY HARRISON P. EDDY,[1] PRESIDENT, AM. SOC. C. E.

It is fitting for organizations as well as for individuals to have periods of review and introspection. Although Engineering has been recognized for a long time as a profession, it may be well to begin by considering the definition of a profession.

One of the most accurate and appropriate definitions is that offered in their report on Professional Associations, by Sidney and Beatrice Webb,[2] British investigators, as follows:

> "A profession is a vocation founded upon specialized educational training, the purpose of which is to supply disinterested counsel and services to others, for a direct and definite compensation, wholly apart from expectation of other business gain."

It is pertinent to inquire whether Engineering as practiced to-day can qualify as a profession under such a definition: Is it founded upon specialized educational training? and is its purpose the rendering of disinterested counsel and services for a direct and definite compensation, wholly apart from expectation of other business gain?

While there are wide differences in engineering activities, some lying strictly within the definition of an art, and others within that of a science, there cannot be any doubt that Engineering is both an art and a science. From both points of view Engineering depends in large measure upon the sciences.

The utilization of the sciences to-day may be illustrated by the work of the Sanitary Engineer. In the science of hydrology, he finds the bases for estimating the quantity of water which he can obtain from an available source. He calls upon the science of chemistry for determination of characteristics, such as the hardness and corrosiveness of the water; colloidal chemistry for ascertaining its behavior with respect to the use of coagulants in purification processes; biology with regard to the effect of floating and other plant and animal life, on the taste and odor of the water; and bacteriology for detecting the probability of the presence of pathogenic infection. In the field of design, he calls to his assistance the science of hydraulics for determining the sizes of aqueducts, pipe lines, and channels; of mechanics for planning safe structures; of metallurgy for aid in the selection and preparation of the most suitable metals and alloys of metals to serve his purposes; and of the relatively new science of soil mechanics to provide the means for more accurate, economical, and safe design of the dams required to form his reservoirs. These are not all the sciences commandeered by the engineer practicing in this field, but a sufficient number have been mentioned to demonstrate the part of science in the intelligent solving of problems in Sanitary Engineering. Similar conditions prevail in all other branches. Any engineer of middle age will recall the trend during the current century toward a greater use of many of the sciences.

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[1] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[2] *The New Statesman,* April 21, 1917, London, England.

This review of the requirements of engineering practice is adequate to demonstrate that it is founded upon specialized educational training.

The question whether Engineering is practiced for the purpose of supplying disinterested counsel and services, can be answered best by considering the character of the work and the manner in which it is being done.

Engineers frequently function as consultants to clients and may be compensated upon a per diem or lump-sum basis, their services being terminated when their advice has been given. Engineers in private practice, as distinguished from those functioning as consulting engineers, generally perform a more extended service, usually including investigation, report, design, and supervision of construction. Their compensation may be a percentage of the cost of construction or a lump sum.

Then, there are the engineering staffs of consulting engineers, engineers in private practice, the Government and its political subdivisions, the industries, and the public service corporations. Large numbers of engineers are devoting their time almost wholly to rendering engineering service for such employers and are compensated upon a salary basis.

Professors and other teachers comprise an important group whose primary function is teaching Engineering and showing the student how to apply the sciences to his problems. Rather than directly rendering engineering service, the teacher usually instructs the pupil how to render such service and generally is compensated by an annual salary.

A large group of engineers perform engineering services primarily for the purpose of selling materials and equipment. Their functions include explaining the qualities, characteristics, and performance of their goods and devising ways in which they can be utilized to better advantage than other goods adaptable to like purposes. Their compensation may be in the form of fixed salaries, although in some cases it may take the form of a commission on the price of goods sold through their efforts.

These illustrations indicate that most engineers are supplying disinterested counsel and services and that a large proportion are rendering services wholly apart from expectation of business gain, other than the direct and definite compensation received for the specific service rendered.

This consideration of the work of the engineer and the manner in which he renders service and is compensated, leads to the conclusion that Engineering as practiced to-day should be classified as a profession, and it truly is a learned profession. It may be well here to emphasize the fact that the engineer who is employed in any of the groups mentioned, and in others, qualifies under this definition and is practicing as a professional engineer just as truly as an engineer who conducts a private practice.

Education

The trend toward greater utilization of the sciences has been accompanied by a necessary increase in the engineer's knowledge. It is natural that this necessity should be accompanied by a pronounced trend toward greater scholastic training, for it is with difficulty that an engineer can acquire by himself a well-rounded education in the sciences. Education, however, does not necessarily include scholastic training, for the records of some of those who have not had the benefit of it demonstrate that ability, application, and study make it possible to accomplish remarkable results without such training.

The development of systematized knowledge in the past generation has been so great and is proceeding at such a rapid rate, that the most efficient standard college course cannot furnish complete mental equipment for the engineer; hence, the present trend away from specialization toward concentration upon the fundamental principles of Engineering and allied sciences, during the regular college course.

Post-graduate work has become a necessity for one who is to secure more than mediocrity in Engineering. Such work for a time may take the form of college training, but its principal part will consist of continuation study. As Dean Robert E. Doherty well states:

> "The principle should be frankly accepted in engineering education that the process is divided into scholastic and post-scholastic stages. * * * The proper time for specialization is after the undergraduate course and in the majority of cases, specialization must accompany active experience rather than take the form of further discipline in college. The provision of effective continuation education for this period remains one of our most urgent problems."

Business and Industry

A moment's reflection will show that some persons work in a dual capacity, exercising both engineering and business functions. These functions are not always easy to separate, there being in many cases a "twilight zone" in which both are merged.

This dual type of functioning may be illustrated by the engineer who is engaged in contracting. One of the most marked trends is toward the entrance of the engineer into contracting, a change which is destined to elevate this business to a higher plane, to the benefit of the public. The contractor may function as an engineer in applying the laws of science to his methods of handling work and he may design important devices and equipment for his use in carrying on his construction. In such ways, he is performing engineering services on his own account and, less directly, for the benefit of his client. His business function includes management, purchasing materials, organizing men, and financing work. These two functions may be equally important, or the engineering may be paramount in some cases and the business function in others.

The older generation has witnessed the growth almost from birth to enormous proportions of industrial and other similar corporations which operate in the field of Engineering, but not being individuals they do not qualify as members of the profession.

Many manufacturers have recognized the valuable assistance to be obtained from the application of the sciences to their processes, equipment, and products, and have employed engineers for such work. In this way products have been improved and often, at the same time, processes and equipment so modified that their cost has been reduced. Thus has industry rendered a most valuable service to the community. This development has created a new field for the professional engineer, without whose knowledge and ability to apply it to problems in design and construction, industry could not have gone forward as it has. As in contracting, so in industry, the entrance of the engineer has wrought great changes and raised this business to a higher and more useful plane.

Then, too, the engineer engaged in selling materials and equipment, who has performed a most valuable service often in a strictly professional capacity, is a direct development of the industries, largely within the last two decades, and the trend, to-day, is unmistakably toward a greater proportionate functioning of this type of engineer in the future.

However, while recognizing the valuable features of industrial development, it is important also to consider some which have been less fortunate in their results. There can be no doubt that there has resulted a marked tendency toward commercializing engineering practice. In some branches the independent practice of the professional engineer has been pushed almost out of existence by the irresistible advance of commercialized engineering. High-pressure salesmanship leading to the adoption of ready-made designs, equipment, and processes, because easily available, even though not the best suited to the conditions, and inappropriate standardization have followed the march of industrialism. In many cases, the quality of the professional practice of engineering has suffered and the standing of the professional engineer in the estimation of the community has been affected unfavorably.

Notwithstanding much current discussion of the unfortunate effect of mechanical equipment in depriving manual labor of its means of earning a livelihood, there seems to be little doubt that this trend toward the greater use of such equipment will continue and that the field of the professional engineer in industry will grow. It is unthinkable that Man, for all time, must do by the sweat of his brow what he can accomplish by the use of machinery and without material exertion.

STANDARDIZATION

Standardization is the direct result of efforts to produce in quantity for the purpose of reducing cost. In many cases it has been of great value. It has made possible the enjoyment of goods and facilities which would have been impossible without it, particularly in the field of industry. On the other hand, the unmistakable trend toward standardization in the field of professional practice in many cases is a distinct handicap to the rendering of professional service of a high order and to the advancement of the science and art of Engineering.

While advantage should be taken of experience, the following of precedent irrespective of its suitability is most unfortunate. An eminent engineer once wrote: "Would it not be a fine thing to set up the principle that no work on which new and advanced experiment and investigation was not carried on and reported could be considered as a real job well done?"

SPECIALIZATION

The older generation has witnessed a most important development of specialization among physicians until now there is some danger that the long-esteemed family doctor will become extinct, particularly in urban communities. In a smaller degree lawyers have specialized until now there are criminal, corporation, taxation, and other lawyers whose major practices are in their particular fields. It has been stated that one must employ a country lawyer if he wishes a well-rounded adviser, because, while such a lawyer has not had sufficient work in any field to enable him to specialize, the diversity of his problems has obliged him to practice in many branches of the law.

The engineer, however, probably has carried specialization beyond that of other professions, first by division of practice into the great branches, Civil, Mechanical, Electrical, Mining and Metallurgical, and Chemical Engineering; then into such sub-branches as railroad, highway, water supply, sewerage, structural, irrigation, and many others of the Civil Engineering group.

This tendency undoubtedly arose from two conditions: First, an engineer who entered a particular field became so engrossed in his work that opportunity to diversify his practice was lacking; and, second, the demand created by the ever-increasing knowledge of his line of activity deprived him of time and mental capacity to acquire the special knowledge necessary for him to qualify in other branches of his profession.

For some years engineering colleges endeavored to respond to the apparent need for highly specialized training, in order that the student might be fitted upon graduation to take up work in a predetermined specialty. Happily this policy seems to be giving way to one of greater emphasis upon the fundamentals, leaving specialization for a later period.

Such specialization as has been described probably is necessary and desirable; with increase in the accumulation of knowledge it will become more pronounced. There is, however, a further degree of specialization, the benefit of which to the individual is not so clear, if indeed it is not an assured misfortune. This is the specialization prevalent upon extensive work and in large engineering organizations where one who shows ability in a certain narrow zone is kept continuously upon problems of substantially the same kind. This course undoubtedly produces the most economical immediate production, but may it not work injury to the employee who thus is deprived of opportunity to occupy a broader field?

RESEARCH

One of the most noteworthy trends in Engineering is toward greater utilization of research, the development of which as a practical aid in the teaching and practice of Engineering began with the adoption of the laboratory method of instruction in the latter half of the Nineteenth Century. It was then and has continued to be realized that "rule-of-thumb" and experience methods are not always founded upon a correct understanding of the scientific principles involved.

Research, however, did not take on its present importance until it was recognized as a tool of industry. The earliest great development naturally came in the field of Electrical Engineering where methods based upon observation, trial-and-error and "horse sense" were handicapped by the illusive nature of electric currents which, in many cases, could not be detected directly by any of the senses. New methods had to be devised, and it was soon found that the results of research were most valuable assets of the electrical industry. Simultaneously, and later, other industries saw the value of research and it became general. More recently the Federal Government through its many Bureaus has inaugurated a policy of basing much of its engineering work upon the results of research.

That the trend is toward a greater dependence upon research cannot be gainsaid, but one may ask, where is it leading? It may lead to greater specialization in practice; it certainly will lead to solving engineering problems with greater accuracy and precision, and it will place Engineering upon a more scientific foundation.

EMPLOYEES

One of the most pronounced differences between the profession of Engineering and those of Law and Medicine is that a large proportion of the engineers are employees, whereas the practices of the lawyers and the doctors in large measure constitute independent professional activities. The trend toward a preponderance of engineering employees is not new. The Webbs have described the condition as follows:

> "* * * the members of the more modern professional associations (including those of engineers) are not independent producers but are employed at stipends, salaries, or wages with instruments of production belonging to their employers, who, whether individuals or corporations, whether aiming at their own profit or at the common service of the community, themselves direct the industry or vocation, (and) control the product,* * *".

The railroad companies early employed large numbers of engineers, and other public utilities more recently have done likewise. Industries for many years have employed engineers, but there has been a rapid growth in their numbers within the last two decades, and the trend seems to be in this direction; but, second only to the industries, the public or governmental service probably has increased most rapidly in the proportion of employed engineers. This has been due in part to the increase in the use of applied science in governmental undertakings and perhaps, in large part, to the entrance of Government into fields hitherto developed by private enterprise. Take, for example, the field of transportation; when the railroads were built it was by private enterprise and capital, but when this field was broadened by the advent of the automobile, Government undertook to build the ways upon which it was to run.

This trend toward an increase in the proportion of professional engineers who are employees, which began in a small way a generation or more ago, is increasing at an accelerated rate. At present, probably about 90% of the members of the Society are salaried employees.

Before leaving the trend toward increase in the great group of engineering employees let me state my conviction that the continuance of Engineering as a profession, with those characteristics to which a profession should adhere, depends largely upon their zeal in taking advantage of every opportunity to increase knowledge and to render disinterested counsel and services; in their hands lies the future of this great profession.

Technical Engineering Societies

Finally, the trends in Engineering Societies may be considered. One is confronted with and appalled by the great number of such societies, some overlapping the functions and duplicating the work of others. There can be no doubt that there is an economic waste due to such duplication of effort. There are some compensating advantages, however, among which may be mentioned development of the interest and a corresponding participation of a greater number of engineers in society work, a greater opportunity for intimate discussion of the work of specialists, and enlarged facilities for reaching engineers in all walks of life. On the other hand, it can be argued with considerable force that real co-operation and a consolidation of effort would result in material benefits to individual engineers and to the profession as a whole.

Social Aspects of Engineering Societies

These considerations apply primarily to the technical branch of society work, but the trends in the social or welfare branch, sometimes called the professional branch, are no less marked. The engineer is interested more keenly and properly than ever before in his personal and vocational welfare. The history of the Institution of Civil Engineers, that honorable and venerable pioneer among societies of professional engineers, is particularly illuminating as written by the Webbs, as follows:

> "The civil engineers desired, in fact, to form a profession, having its own technique, its own code of manners and its own solidarity against other professions and the public. * * * The constitution of the Institution of Civil Engineers was modelled on that of a learned society; * * *.

> "For the first forty years the Institution remained nothing but a scientific or technical society, providing opportunities for exchanging professional information, * * * . This function still remains the dominant one. But during the past half-century the Institution has gradually taken on the task of improving the status and protecting the interests of its members. * * * Out of these investigations arose a demand for greater strictness with regard to entry into the Institution itself and indirectly, into engineering practice."

In the United States during the past fifteen years there has been an increasing trend in the latter direction. This is shown by the adoption of registration laws in many of the States, now numbering twenty-eight, for the purpose of protecting the health and assuring the safety of the public with respect to engineering works, but indirectly tending to restrict practice to engineers who have certain prescribed qualifications.

Perhaps the most marked trend in the social branch of society work has been the increasing interest and effort in aiding engineers to obtain employment during the current period of depression. Never before have Engineering Societies taken such interest in the welfare of engineers. These activities should make for a greater spirit of loyalty and co-operation.

Another important trend has been toward a larger participation in governmental affairs, with a view to securing a greater degree of excellence in the design and execution of public works.

To sum up the trends, one must include a more certain professional status founded upon specialized educational training and the rendering of disinterested counsel and advice; greater need for scholastic and continuation education; more engineering in industry and a greater influence of industry upon the practice of engineering; a greater degree of standardization; more specialization; more dependence upon research; an increasing number of engineer employees; and greater activity of Engineering Societies particularly in the field of social activities.

It would be presumptuous for me to comment specifically upon trends in Engineering as a profession in Canada, but I would not be loyal to the Engineering Institute of Canada, of which I have the honor of being a member, if I did not give public recognition to the splendid work which it has done in upholding and advancing the profession.

In the field of Engineering the trend is unmistakably toward better things for the community, the profession, and the individual engineer, to be secured in large measure by the co-operative effort of engineers made effective through Engineering Societies. May we not count upon more frequent and intimate co-operation of the Engineering Institute of Canada and the American Society of Civil Engineers?

MEMOIRS OF DECEASED MEMBERS

WILLIAM CAWTHORNE UNWIN, Hon. M. Am. Soc. C. E.[1]

Died March 17, 1933

William Cawthorne Unwin was a descendant of a Hugenot ancestry which had been long established in the Eastern Counties of England. The branch to which he belonged had been settled at Coggeshall, a small country town in Northern Essex, near the borders of Suffolk, where his father, William Jordan Unwin, was born in 1811. William Jordan Unwin early in life decided to enter the Congregational ministry and, after studying at the Highbury Training College, London, England, and at Glasgow University, Glasgow, Scotland, took up his first pastorate at Woodbridge, Suffolk, in 1835. He married Eliza Davey, daughter of J. Bailey Tailer, of that town, and his eldest son, William Cawthorne Unwin, was born at Coggeshall, on December 12, 1838. Later, the Rev. W. J. Unwin took up the educational side of his profession and, ultimately, in 1852, was appointed Principal of Homerton College, Homerton, then a village outside London, but long since absorbed into the industrial eastern area of the Metropolis. This appointment was, in those days, the highest educational preferment in the gift of the Congregational Board. Dr. W. J. Unwin—he was made a Doctor of Laws by Glasgow University, in 1864—established a considerable reputation as an educationalist, and his work is referred to very highly in the Government reports of the time.

William Cawthorne Unwin received his early education at the City of London School. Probably, in the normal course, a boy of his ability and position would have proceeded thence to one of the Universities. In the middle of the Nineteenth Century, however, such a course was impossible to one who did not subscribe to the tenets of the Church of England. Accordingly, Unwin studied as a lay student at New College, St. John's Wood, London, which was formerly a college for training for the ministry.

In 1856, at the age of seventeen, he obtained, through the instrumentality of Mr. Tate, a mutual friend, an introduction to that famous British engineer, William Fairbairn, of Manchester, England (1787-1874). Fairbairn was in need of what he described in a letter to the Rev. W. J. Unwin as "a clerk or secretary" who would assist "in drawing up reports and in other literary and scientific pursuits." Thus commenced the association between two outstanding figures in engineering progress, which was only broken by the death of Fairbairn. Their two lives cover a period from the present day almost to the time of the founding of the United States, and to the year of Fairbairn's birth when the storming of the Bastille inaugurated that outburst of smouldering discontent which led to the French Revolution and so profoundly affected the history of the civilized world. Mr. Unwin lived to see the struggles of a disunited but far greater world in its attempts to recover from a war, compared with which the struggles of the Napoleonic wars in Fairbairn's youth were insignificant.

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[1] Memoir prepared by E. G. Walker, M. Am. Soc. C. E.

When Mr. Unwin joined him, Fairbairn was at the height of his career. He was the head of a large and famous engineering works, fully employed on the manufacture of power plant, both steam and water, and the construction of the mills and factories which were being established over the north of England at that time. He was also keenly engaged on a number of researches, for which his practical mind continually saw the need in the lack of scientific data available for the solution of the fresh engineering problems, which were arising with increasing rapidity. In Mr. Unwin he found as Assistant whose serious, scientific, yet practical, character eminently fitted him to carry out the details of these researches, and the most cordial relations existed between the two from the outset. In after years, Mr. Unwin frequently spoke in terms of high respect and unconcealed affection of his "old chief" under whom he started his engineering career.

Mr. Unwin remained in Fairbairn's service until 1862. During this period he was engaged on a number of investigations, and assisted Fairbairn in his practical problems as well. The first research of importance was that upon the properties of saturated steam, which Fairbairn had originally projected because of the lack of working data then (1856) available. The results of this work were published in a paper by Messrs. Fairbairn and Tate read before the Royal Society in 1866.[2] Another research upon which Mr. Unwin was engaged, was that described by Fairbairn in a paper[3] entitled "On The Resistance of Tubes to Collapse."

In 1859, iron began to replace wood as a material for constructing warships, although iron merchant ships had been built sixteen years earlier. At the beginning, little was known of protective armor for iron ships, and the British Admiralty, in 1860, appointed a Special Commission to study this subject. Fairbairn was one of the civilian members and Mr. Unwin, as his Assistant, took such a large part in its work that he was thanked officially at the conclusion of the study and awarded a gratuity by the Admiralty.

This period was also the time of the first introduction of continuous brakes on railway trains. There was keen competition between two systems, the Newall, introduced on the East Lancashire Railway, in 1853, and the Fay, which was an improvement on the original Newall type, and was introduced on the Lancashire and Yorkshire Railway, in 1857. In 1859 Fairbairn was called upon to make comparative trials of these brakes for the Lancashire and Yorkshire Railway Company. Mr. Unwin carried out the details of the work, which were described in a report to the Railway Company, prepared mainly by Mr. Unwin, and also in Fairbairn's paper read before the Institution of

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[2] "Experimental Researches to Determine the Density of Steam at Different Temperatures and to Determine the Expansion of Super-Heated Steam," by William Fairbairn and Thomas Tate, *Philosophical Transactions*, Royal Soc., Vol. CL (1860), p. 185; also, "Experimental Researches to Determine the Density of Steam at Various Temperatures," by William Fairbairn and Thomas Tate, Rept., British Assoc. for the Advancement of Science, 1859, p. 253.

[3] *Philosophical Transactions*, Royal Soc., Vol. CXLVIII (1858), p. 389.

Civil Engineers in 1860.[4] The design of the Fay and Newall brake which was evolved subsequently, was influenced by suggestions which Mr. Unwin put forward as the result of these trials. This brake was largely used on the British railways until it was completely superseded by the pneumatic brake which the late George Westinghouse, M. Am. Soc. C. E., began to develop a decade later.

One of the most important of the researches, by reason of its far-reaching consequences, on which Mr. Unwin assisted Fairbairn about this time, was that on the effect of repetition of loading on the strength of wrought-iron girders. In those days wrought iron was beginning to compete against cast iron for the construction of railway bridges, but little was known about the qualities of the new material. The Board of Trade, which controlled the British Government requirements for railway construction, had established, as the result of a report of a Royal Commission in 1847, rules for the breaking weight of cast-iron bridges, but nothing had been done to settle the admissible loading for bridges of wrought iron. The first definite suggestion, made as the result of a dispute about the safety of a wrought-iron bridge built at Torksey, in Lincolnshire, was that the maximum stress should be limited to 5 tons per sq in. In 1858, Fairbairn constructed another wrought-iron bridge over the River Spey, in Scotland. As the result of disagreement about this bridge, with the Inspector for the Board of Trade, he initiated, with Mr. Unwin's assistance, a research, in 1860, on a full-sized girder which he subjected to various repetitions of load. The research[5] was completed in 1862 and reported to the Royal Society in 1863-1864. These were the first systematic experiments on fatigue. The results are summarized in Fairbairn's paper, as follows:

> "From these experiments it is evident that wrought iron girders of ordinary construction are not safe when submitted to violent disturbances equivalent to one-third the weight that would break them. They, however, exhibit wonderful tenacity when subjected to the same treatment with one-fourth the load; and assuming, therefore, that an iron girder bridge will bear with this load, 12 million changes without injury, it is clear that it would require 328 years at the rate of 100 changes a day before its security was affected. It would, however, be dangerous to risk a load of one-third the breaking weight of bridges of this description, as, according to the last experiment, the beam broke with 313 000 changes; or a period of 8 years at the same rate as before would be sufficient to break it. It is more than probable that the beam had been injured after the previous 3 million changes to which it had been subjected; and assuming this to be true, it would follow that the beam was undergoing a gradual deterioration which must sometime, however, remote, have terminated in fracture."

Thus, practical design conclusions were established several years before Wöhler initiated his famous elaborate laboratory experiments which were the forerunners of the extensive scientific research on this subject which is being continued at the present day.

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[4] "Efficiency of Various Kinds of Railway Brakes; with Experimental Researches on Their Retarding Power," by William Fairbairn, *Minutes of Proceedings*, Inst. C. E., Vol. XIX (1860), p. 460.

[5] "Experiments to Determine the Effect of Impact Vibratory Action and Long Continued Changes of Load on Wrought Iron Girders," by William Fairbairn, *Philosophical Transactions*, Royal Soc. Vol. CLIV (1864), p. 311; also, "Experiments to Determine the Effect of Vibratory Action and Long Continued Changes of Load upon Wrought Iron Girders," by William Fairbairn, Rept., British Assoc. for the Advancement of Science, 1860, p. 45, and 1861, p. 286.

While working with Fairbairn, Mr. Unwin had studied privately, and, in 1861, he sat for and obtained the degree of Bachelor of Science from London University. Under modern conditions this would hardly be a noteworthy incident, and its significance can only be appreciated fully by recalling the relation between pure science and engineering three-quarters of a century ago. At that time, scientific study of engineering hardly existed, and the fact that Mr. Unwin attached such importance to a degree in science as to devote his leisure to qualifying for it, surely indicates in the young man of 23 firmly rooted views of the value of the scientific training of engineers, in which he took so pre-eminent a part in later years.

In 1862, Mr. Unwin was appointed Works Manager to Williamson and Brothers, of Kendal, Westmoreland County, England. This firm was one of the general engineering establishments of the period, making mill machinery and water-power plant. The Messrs. Williamson were at that time sole makers of the Thomson vortex turbine, an inward-flow type developed by Professor James Thomson, of Belfast, Ireland, brother of the late Lord Kelvin. Professor Thomson had been a pupil of Fairbairn and had first designed his vortex turbine (in which the runner was surrounded by a chamber set eccentrically to it), in 1851, about the same period as the late J. B. Francis, Past-President and Hon. M. Am. Soc. C. E., was experimenting with his early inward-flow turbines, at Lowell, Mass. At Kendal, Mr. Unwin was engaged on the practical work of designing and constructing turbines, and in mill work, but he found time to continue his studies of the research problems on which he had worked with Fairbairn. The two kept in constant touch, and Mr. Unwin assisted Fairbairn in the revision of some of his engineering textbooks.

At the beginning of 1866, Mr. Unwin returned to Manchester, where he became Manager of the Engine Department of the Fairbairn Engineering Company, starting a second period of service under his old chief. This time he was in a more responsible position, although, in the intervals of his business occupations, he was still able to devote attention to the research side. In 1867, he was elected an Associate of the Institution of Civil Engineers, being proposed by Fairbairn. In this year, he competed unsuccessfully for the appointment of Professor of Engineering at Owen's College, Manchester, which was then in the early stages of its development.

His first active work as a teacher began in the spring of 1868. While the Owen's College professorship was still being discussed by the Senate, he was selected as a Special Lecturer to give a course of lectures to the officers of the Royal Engineers, at Chatham, England. His entry into this new work was an immediate success and, in due course, he was called upon to give several more courses. Between 1868 and 1871, he gave five series of lectures covering bridges, structural design, turbines and hydraulic machinery, the steam engine, and railway construction. His first series was published, with extensions, in book form, and "Wrought Iron Bridges and Roofs," published in 1869, was the first of his engineering textbooks.

In 1868, Mr. Unwin was appointed Instructor in Marine Engineering at the Royal School of Naval Architecture and Marine Engineering, at South Kensington. This school was a Government institution, established jointly by the Admiralty and the Science and Art Department (since merged into the Board of Education) in 1864, and was absorbed into the Royal Naval College at Greenwich in 1873. It numbered among its students some of the most famous constructors and engineers in the British Naval Service, including the late Sir William H. White, Hon. M. Am. Soc. C. E., and Sir Phillip Watts, two of Britain's most famous Chief Constructors. Mr. Unwin taught at the school for four years, during which time he continued his lectures at Chatham.

In 1870, the British Government established at Coopers Hill, near Staines, a college for the training of engineers for the public services in India. This establishment, known as the Royal Indian Engineering College, was opened in 1871, and Mr. Unwin was appointed Professor of Hydraulics and Mechanical Engineering, his old friend and senior, Calcott Reilly, being the Professor of Construction. Among the other professors whose names subsequently became famous throughout the English-speaking world, were Wolstenholme, the Professor of Mathematics, and Greenhill, the Professor of Applied Mathematics. At Coopers Hill, began the second phase of Professor Unwin's career. As an Assistant to and Collaborator with Fairbairn, he had established a sound reputation as a scientific engineer and teacher among those with whom he came into contact, but he had not commenced the active independent work which ultimately brought to him such full recognition in British, American, and European engineering circles. He held his professorship at the Royal Indian Engineering College for twelve years, from 1872 to 1884, until he was called upon to undertake the greatest of his educational appointments in the inauguration and development of the City and Guilds College at South Kensington. His first work outside his college duties at Coopers Hill was the finishing for Fairbairn of his last paper[6] "On the Durability and Preservation of Iron Ships and on Riveted Joints," which Fairbairn found himself, owing to advancing age, unable to complete. Fairbairn died the following year at the age of 87.

In 1874, Professor Unwin contributed a number of articles on structural subjects to "Spon's Dictionary of Engineering," an engineering encyclopædia which was much used by British engineers half a century ago. In 1876, he read his first paper before the Institution of Civil Engineers, "The Resistance of Boiler Flues to Collapse."[7] About eighteen years before, he had assisted Fairbairn in the experimental investigation which had resulted in the latter's paper[3] on the subject. The conclusions then arrived at had been accepted generally as the basis of flue design and construction, and several attempts had been made to put the results in a form suitable for mathematical computation without success until Professor Unwin re-examined them and published the formulas which have since been universally accepted. In the same year, the Institution of Civil Engineers published his paper[8] on "The Motion of Light Carriers in Pneumatic Tubes," in which he developed the theory of a subject that was being brought into prominence by the development of the pneumatic method of transmitting telegrams between post-offices in London, Paris, France, and other large cities.

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[6] *Proceedings*, Royal Soc., Vol. XXI (1873), p. 259.

[7] *Minutes of Proceedings*, Inst. C. E., Vol. XLVI (1876), p. 225.

Shortly thereafter he was engaged in the preparation of his two most widely known books. The first edition, in one volume, of his "Elements of Machine Design," was issued in 1877. Its subsequent history surely justifies its inclusion among the classics of engineering, for few technical works have had such a widespread influence on the details of engineering design. During the half century which has elapsed since its first publication, it has passed through more than thirty editions and has served as a textbook for countless classes in British and American engineering schools, as well as being a generally used reference book in drafting offices. A French edition was published in 1882, and a German edition in 1885. In 1890, the subject-matter was divided into two volumes. In later years, the work of revision was undertaken by Professor A. L. Mellanby, of Glasgow.

About the same time as he initiated his "Machine Design," Professor Unwin undertook the preparation of the article on "Hydraulics" for the Ninth Edition of the Encyclopædia Britannica, which was published in 1881. This article became a standard textbook on the subject. It was used as a basis for design by the leading British water engineers for many years. It was also the accepted technical authority for engineers engaged in the extensive irrigation works which were initiated in India during the latter part of the Nineteenth Century, and was quoted in the Courts in settlement of water disputes. In 1907, Professor Unwin prepared a revised "Treatise on Hydraulics," embodying most of the material of the original article, and also the results of his studies of the mechanics of compressible fluids which he had started in 1874. To the re-written Eleventh Edition of the Encyclopædia, published in 1911, Professor Unwin contributed articles on "Hydraulics" and "Bridges."

In 1878, Unwin was transferred to full membership of the Institution of Civil Engineers, and was elected a Member of the Institution of Mechanical Engineers.

Toward the end of Professor Unwin's connection with the Royal Indian Engineering College, the Metropolitan Board of Works for London undertook the re-organization of the sewerage system of the Metropolis. A committee was appointed to study the subject, and Professor Unwin undertook for it an extensive investigation on the tidal flow in the River Thames, which resulted in considerable modification of the original proposals for the discharges from the main outfall sewers.

In the late Seventies, a strong movement sprang up among the ancient Livery Companies, or Trade Guilds, of the City of London, which resulted in the establishment, in 1878, of a body composed of representatives of the Corporation of London and of the principal Companies — the Goldsmiths, Clothworkers, and Mercers predominating—known as The City and Guilds of

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[8] *Minutes of Proceedings*, Inst. C. E., Vol. XLIII (1875), p. 263.

London Institute. This body was charged with the utilization of funds subscribed by the Corporation and Companies for the development of technical education. It began by establishing the Finsbury Technical College and, later, a Crafts School for artisans and apprentices in South London, and a general project for technological examinations; but its principal activity was the founding of the Engineering College at South Kensington, known, successively, as The Central Institution, The Central Technical College, and (now) The City and Guilds (Engineering) College. When the Imperial College of Science and Technology was founded at South Kensington in 1907, by Royal Charter, the City and Guilds College was incorporated therein.

In 1884, when the building of the College was sufficiently advanced, Professor Unwin was offered the Professorship of Civil and Mechanical Engineering, the principal Department. In 1885, the equipping of the College was completed, the first classes were started and Professor Unwin was appointed Dean. He remained the Administrative Head until 1896, when a new system was established whereby the heads of each Department held the appointment in turn. Under this arrangement, the Deanship reverted to him again in 1902, and he held it for two years until his retirement from the College in 1904.

The planning and arranging of the laboratories, workshops, and the curriculum of his Department, devolved completely upon Professor Unwin. In those days—fifty years ago—laboratory teaching of engineers was new and Professor Unwin and his contemporary, the late Sir A. B. W. Kennedy (then Professor at University College, London), can be reckoned among the pioneers of this form of education on the scale on which they started it. Professor Unwin's work was particularly successful, and gave to the Central an initial impulse which soon placed it in the front rank of British engineering colleges.

Although fully occupied with this work, Professor Unwin found time during the first years of his professorship at South Kensington, for a large amount of original work on a wide range of subjects. Between 1885 and 1890, he wrote papers on fluid friction, testing of cements and mortars, properties of steam, power transmission by compressed air, and strength of materials. He also wrote his book, "The Testing of Materials of Construction," the First Edition of which was published in 1888, and the Third (last) Edition in 1910. He carried out a research for the British Association for the Advancement of Science on the variation of strength of alloys with temperature, which was presented in a report[9] entitled "Strength of Alloys at Different Temperatures." In 1887, Professor Unwin carried out various investigations and tests in connection with the building of the Forth Bridge, in collaboration with the famous designer of the bridge, the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E. He was in correspondence with many eminent engineers and investigators on matters relating to the strength of materials, hydraulics, and prime movers. There exists some interesting corespondence between Professor Unwin and the late Clemens Herschel, Past-President and Hon. M. Am. Soc. C. E., in 1888, relative to Mr. Herschel's then new invention of the Venturi meter. Professor Unwin's services were also in request for expert evidence on technical matters before Parliamentary Committees, Courts of Justice, and Abitration Tribunals. His scientific abilities and technical investigations led to his election to Fellowship of the Royal Society of London in 1886.

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[9] Rept., British Assoc. for the Advancement of Science, 1889, p. 746.

In 1890, occurred the event which led to Professor Unwin's direct connection with engineering work in the United States. The project which originated in 1886, a year after his first students commenced work at the Central Institution, for the development of the water power at Niagara Falls, began to take shape. On January 1, 1890, the late Coleman Sellers, M. Am. Soc. C. E., took office as the Engineer to the Cataract Construction Company, of Niagara Falls, N. Y., the organization formed to carry out the work. The magnitude of the project being so greatly in excess of any other that had been developed to that time, as well as the novelty of the proposal to transmit the power to distant points, led to the decision to make the most use possible of the experience available in Europe. Accordingly, the late Edward Dean Adams, F. Am. Soc. C. E., the President of the Company, established himself in London and visited, and collected data from, all available sources in England, France, Switzerland, Germany, and Italy. It was then decided to form an International Committee to consider and advise on competitive plans. After negotiations, initiated in May, 1886, the International Niagara Commission was formed in what afterward became known as "The Niagara Room," in Brown's Hotel, Dover Street, London, on June 21, 1890. The late Lord Kelvin was Chairman, and the members were Professor E. Mascart, of Paris, Dr. Coleman Sellers, of Philadelphia, Pa., Lt.-Col. T. Turrettini, of Geneva, Switzerland, and Professor Unwin. Professor Unwin was appointed Secretary, and the headquarters of the Commission were established at the Central Technical College. The story of the work of the Commission has been told fully elsewhere (see, in particular, "Niagara Power," by the late Mr. Adams). Professor Unwin prepared the report of the Commission, which completed its work in April, 1891. Upon this report the first development of 125 000 hp, at Niagara, was based. The work was carried out by Dr. Sellers, and Professor Unwin and Colonel Turrettini were appointed Foreign Consultants to the Company. In this capacity, Professor Unwin paid several visits to the United States in subsequent years.

His original work, during the next few years, was concerned largely with prime movers. When he equipped the laboratories of the Central Technical College, one of the major pieces of apparatus provided was an experimental steam engine of special design, arranged so that clearance volumes, expansion ratios, and as many as possible of the factors which enter into the economy of the working of a steam engine, could be varied and the effects demonstrated by actual measurement. Professor Unwin's experimental studies of steam-engine economy, and his researches in conjunction with Bryan Donkin and others, resulted in his being called on to test steam plants in different parts of England. When the practical development of oil engines started in the late Eighties, he took an important part in their experimental development, and read a paper,[10] in 1892, on "Petroleum Engines," at the Institution of Civil Engineers, in London, mainly based on his work on the Priestman engine. About the same time, Dr. Sellers read a paper before the Franklin Institute, in Philadelphia, upon the modified American Priestman engine. In subsequent years, Professor Unwin conducted trials of other designs, and in 1897, made a visit to Nürnburg, Germany, where he reported upon the then newly-designed oil engine of Rudolf Diesel, made by the Maschinenfabrik Augsburg-Nürnburg. This report aroused much interest in Great Britain, and had considerable influence upon the development of the heavy-oil engine in that country. An equally momentous visit, in 1892, to Colmar, Alsace, made for the purpose of testing a 500-hp, steam plant working on superheated steam, led to attention being again directed in England to superheating, after a lapse of nearly forty years. In 1893, Professor Unwin and his colleague, W. E. Ayrton, Professor of Electrical Engineering at the Central Technical College, went to Chicago, Ill., to act as Judges representing the British Section of the Centennial Exposition. In 1895, he delivered the James Forrest Lecture of the Institution of Civil Engineers, on "The Development of the Experimental Study of Heat Engines."[11] a subject on which he had done original work from the commencement of his career, nearly forty years before. The lecture was a masterly exposition of the advantages of rational scientific experimentation.

In 1895, the failure of the Bouzey Dam, in France, attracted the attention of engineers to the problems of the stability of such structures. Much work was being done in England about this time on the extension of the water supply of the larger cities and the provision of impounding reservoirs. The building of the Aswân Dam, in Egypt, was also being considered. Professor Unwin took an important part in the discussion of these problems and his advice was sought in connection with a number of these works, including the dams and hydraulic problems of the Liverpool, Birmingham, and Derwent Valley water supplies. In 1892, when on one of his professional visits to Niagara, he reported to the East Jersey Water Company on the adequacy of the Company's arrangements for the supply of water to the City of Newark, N. J., and on proposals for its extension.

Next after the Niagara power project, perhaps the largest individual engneering work upon which Professor Unwin was engaged was that of the Coolgardie water supply, in Western Australia. He was one of a Committee of Three appointed by the State Government to evolve a plan for pumping 5 000 000 gal of water per day over a distance of 328 miles through almost desert country. Although many larger projects are now in successful operation, forty years ago this was a problem of very considerable magnitude.

In 1901, the late Sir John Wolfe Barry initiated what has since come to be one of the most far-reaching engineering developments by reason of its universal adoption. In that year he organized a small committee, known as the British Engineering Standards Committee, the object of which was to simplify technical practice by standardizing some of the principal details of civil engineering work, such as rolled structural sections, rails, etc. Professor Unwin was a member of this Committee and was largely responsible for the detailed calculations which led to the preparation of the first British Standard Sections. For this Committee he also carried out his investigation on the elongation of similar tensile test bars which resulted in the adoption of the standards which have been used in British Standard Specifications ever since. He remained a member of the Main Committee of the organization (now the British Standards Institution) to the end of his life, and served on many Sub-Committees from time to time.

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[10] *Minutes of Proceedings*, Inst. C. E., Vol. CIX (1892), p. 1.

[11] *Loc. cit.*, Vol. CXXII (1895), p. 154.

He retired from teaching in 1904, when he was elected the first Professor Emeritus of the Central Technical College. In 1900, he had been made a member of the Senate of the University of London, and was the first University Professor of Engineering to be appointed when the plan for incorporating the colleges of London into the University came into being.

Professor Unwin's career is remarkable for the fact that much of his important public service was done after his retirement, and, at a period in life, when most men expect some relaxation of the pressure of work. When he retired from his professorship, at the age of 65, however, he commenced a period of nearly thirty years of activity that was almost as full as when he was in the prime of manhood. He refers typically to this in a letter which he wrote later to his old friend, Mr. Edward Dean Adams,

> "I retired some years ago from my professorship and am nominally on the shelf. But I have found as much to do as before, only my work is chiefly voluntary. I have just passed my 80th birthday, but I am in good health with much power of work still left."

As far back to the early years at the Central Technical College, Professor Unwin had been called from time to time to serve on Government committees of inquiry into engineering subjects, and had made a number of investigations in his laboratories in connection therewith; his work in these directions continued to within a short time of his death. He also served on many Government inquiries into new public works projects in India and elsewhere, in addition to Research Committees of the Institutions of Civil and of Mechanical Engineers, but perhaps his greatest work for these Institutions was the predominant part which he played in the development of the standards of educational qualifications and of the examinations for admission which they maintain. Membership in these Institutions is sought increasingly as engineering qualifications in all British countries, so that Professor Unwin's share of the work of determining the educational requirements upon which the Institutions now insist, has had far-reaching consequences in the maintenance of a high standard of technical ability.

In 1905, he was made an Honorary Doctor of Laws of the University of Edinburgh, Scotland, in recognition of his eminent services to Engineering Science, both in its scientific and practical sides.

Reinforced concrete, as a structural material, was first introduced into England in 1899, and, by 1905, its possibilities had been recognized to an extent sufficient to induce the Royal Institute of British Architects to form a committee to investigate it. Professor Unwin, who in 1886 had been made an Honorary Associate of the Institute, served as Vice-Chairman of the Committee, and the mathematical basis of the methods of calculation recommended in the report[12] which was made—and which for many years was the accepted British standard—was the work of Professor Unwin and William Dunn, a joint author of the first English textbook on reinforced concrete, published in 1906. Professor Unwin also served on the Committee on Reinforced Concrete which was subsequently appointed by the Institution of Civil Engineers.

In 1911, he was elected President of the Institution of Civil Engineers. His Presidential Address[13] dealt with the development of two of the subjects to which he had given so much original thought and effort during his career, namely, the knowledge of the strength of materials and of engineering education. A sentence is quoted herein which, although it is Professor Unwin's tribute to an earlier pioneer, yet is in fact a comment which can be applied with all truth to his own work. Speaking of the early English engineer, Thomas Tredgold, he said:

> "He had a practical insight into what was important and what was negligible in engineering problems greater than that of writers with more ample mathematical resources, and he rendered essential services to the engineers of his time."

This same practical insight was characteristic of all of Professor Unwin's own work. He possessed the gift, much too rare among scientific investigators, of attaining results which were capable of direct and immediate application to the practical problems of the engineer.

In the following year (1912), he attended, in New York, N. Y., as the Senior British Delegate, the Sixth Congress of the International Association for Testing Materials, and was awarded a medal by the Congress for his work on testing.

Professor Unwin became President of the Institution of Mechanical Engineers in 1915 and served in that office through two years of unprecedented strain upon the mechanical engineering resources of Great Britain. Although at the date of his election, he had reached his seventy-seventh year, the stress of the World War brought to him a period of heavy work. As a member of the Management Board of the Metropolitan Munitions Committee, and of its Sub-Committee on the Production of Gauges, he shared responsibility for placing contracts involving the expenditure of many millions of pounds. His intimate knowledge of the technical progress of Germany enabled him to see more clearly than most of his associates the material strength of the Central Powers, and his foresight caused him to appreciate the difficulties of post-war reconstruction which since have come to pass. His short Presidential Address [14] at the commencement of his second year of office shows this clearly.

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[12] Rept. of the Joint Committee on Reinforced Concrete, Royal Inst. of British Archts., 1907.

[13] *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXVII (1911), p. 2.

[14] *Proceedings*, Inst. Mech. Engrs., 1915, p. 297, and 1916, p. 579.

In 1917, the Institution of Civil Engineers awarded him the Howard Prize for "classic work on the testing of materials of construction and numerous and valuable investigations bearing on the strength of iron and steel."

In 1921, Professor Unwin attained the highest honor which it is in the power of British engineers to bestow, when he was awarded the first Kelvin Medal. This triennial award was instituted as a memorial to the late Lord Kelvin, and is controlled by a Joint Committee representative of the major British Engineering Institutions. The medal is given for outstanding eminence in engineering, and representative bodies in all parts of the world are consulted. It is of interest, that five years later, the John Fritz Medal was awarded to Professor Unwin's old friend, the late Mr. Edward Dean Adams, with whom more than thirty years before he had worked on the great pioneer enterprise at Niagara Falls.

The American Society of Civil Engineers conferred its Honorary Membership on Professor Unwin in 1922. It was appropriate that this honor should have been conferred at the same time on his old friend and co-worker in the field of hydraulics, the late Mr. Clemens Herschel. In 1924, he was made an Honorary Member of the Institution of Civil Engineers, an honor the more unique in that it has rarely, if ever, been conferred on a Corporate Member.

From time to time after the war, Professor Unwin was consulted on special engineering problems. He continued also to serve on the various scientific boards and committees with which he had been connected previously. After 1924, however, he gradually relinquished these, although as late as 1932, he was consulted by the British Standards Institution on certain hydraulic matters. One of the last engineering works on which his advice was sought was the new Mersey Tunnel between Liverpool and Birkenhead, which was opened in 1934. He reported, in 1922, to the Consulting Engineers on questions relating to the strength of the tunnel lining.

During the last years of his life, declining health and failing eyesight prevented Professor Unwin from taking further part in active work. He lived quietly in the flat in Kensington that he had taken on his appointment to the Central Institution in 1884, and which he had occupied continuously for nearly half a century. He was a bachelor. Nevertheless, he had a love for children and easily won their confidence. A natural shyness and diffidence resulted in an aloofness of manner through which only a few penetrated to the richness of character and unassuming ability that lay beyond. He won the regard of his students by the strength of his personality and the sincerity of his interest in them, not only in their college days, but during the whole of their careers, and he esteemed the regard of his old students as his greatest reward. The eminence of his attainments resulted in his being frequently consulted upon engineering works of the first magnitude, but he would never allow the call of outside work to interfere with his college duties.

Professor Unwin was an intense thinker, with more than ordinary powers of concentration, but this was balanced by ability to relax and to enjoy the humorous side of things. He was a man of wide culture and took an interest in many branches of science and art. He was an enthusiastic photographer from the days of wet plates onward, a keen fly-fisher, and, in his younger days, accomplished a great deal of mountain climbing in Switzerland and other European countries. He was a member of the Reform, Savile, and Whitehall Clubs, in London.

Professor Unwin died on March 17, 1933. The funeral service was attended by many representatives of his old students and of the societies and other bodies with which he had been connected. In the funeral oration, his friend, the Rev. Dr. Thomas Yates, paid an eloquent tribute to his character:

> "There was in Dr. Unwin a delicacy of mind and a most winning humility * * *. The last thing he would himself speak of was himself, and he would put aside with quiet but firm dismissal all personal reference from others * * *. His keenness in his calling had no concern in it for material rewards * * *. He served his calling as a devout and true man serves his faith, with a pure mind, a deep devotion and with the humility which learns to the end. His one pride was in the achievements of those whom he had taught, and his happiness was unfeigned when honors came to his contemporaries and colleagues * * *."

Professor Unwin obtained his degree of Bachelor of Science at the University of London in 1861; he was made an Honorary Doctor of Laws of Edinburgh University in 1905. He became an Associate of the Institution of Civil Engineers in 1867, a Member in 1878, a Member of Council in 1909 (retiring in 1926), a Vice-President in 1908, President in 1911, and was elected an Honorary Member in 1924. He joined the Institution of Mechanical Engineers as a Member in 1878, was elected an Honorary Life Member in 1896, served as a Member of Council from 1919 to 1923, was a Vice-President in 1912, and President in 1915 and 1916. He was elected a Fellow of the Royal Society of London in 1886, and served on its Council from 1894 to 1896. He was made an Honorary Associate of the Royal Institute of British Architects in 1886, an Honorary Member of the Franklin Institute, and of the American Philosophical Society in 1890, and of the American Society of Mechanical Engineers in 1898. In 1888, he was President of the Junior Institution of Engineers (London), and, in 1892, of the Mechanical Science (now Engineering) Section of the British Association for the Advancement of Science. He was elected a Member of the Royal Society of Arts in 1877, and served on its Council for the 1912-13 Session. He was President of the Smeatonian Society of Civil Engineers, which claims to be the oldest society of engineers in the world, in 1913. He served as a member of the Main Committee of the British Standards Institution from its formation in 1900 until his death. He was appointed a member of the General Board of the National Physical Laboratory on its formation in 1900, resigning from this in 1928. He was a member of the Senate of the University of London from 1900 to 1905 and, again, from 1911 to 1923. He was a member of the Governing Body of the Imperial College of Science and Technology from 1910 to 1926, and of the Delegacy of the City and Guilds College from 1911 to 1926. He was elected a Membre d'Honneur de l'Association des Ingenieurs sortis de l'Ecole de Liège, in 1922.

Professor Unwin was the author[15] of numerous papers, lectures, and addresses to engineering societies, principally the Institutions of Civil and of Mechanical Engineers, but he also contributed to the Royal Society, the Physical Society, the Society of Arts, and others.

He received many awards for his papers and investigations, including the Telford Medal and the Telford Premium (1876), the Watt Medal (1877), the George Stephenson Medal (1897), the Howard Prize (1917), the Kelvin Medal (1921), all from the Institution of Civil Engineers, the Medal of the Society of Arts (1886), and the London Medal of the Institution of Gas Engineers (1904).

Professor Unwin was elected an Honorary Member of the American Society of Civil Engineers on June 19, 1922, in recognition of the world-wide greatness and influence of his achievements in the practice and teaching of civil engineering.

BASIL CONDON BATTYE, M. Am. Soc. C. E.[1]

DIED MAY 18, 1932

Basil Condon Battye was the youngest son of the late Major Leigh Richmond Battye, of the Indian Army, and Margaret Moffat, daughter of General A. K. Moffat, of the Bengal Staff Corps. He was born at Doonga Gali, on the Northwest Frontier of India, on September 24, 1882, and was educated at a preparatory school in Clifton, England, and, later, at St. Lawrence College, Ramsgate, whence he passed into the Royal Military Academy, at Woolwich, in 1899. In 1900, while yet under 18 years of age, he obtained his commission in the Corps of Royal Engineers having been graduated at the head of the cadets of his year.

After spending three years at the School of Military Engineering, at Chatham, England, Lieut. Battye went on active service to Somaliland, in Africa, where he helped to organize the water supply on the lines of communication—a very difficult problem in that country. He was awarded the African General Service Medal and Clasp at the end of the operations in 1904.

After another period of duty at Chatham, he was sent to India where he was posted to the Military Works Services at Ferozepore and Dalhousie, Punjab. When he was at Ferozepore a serious fire broke out in the ammunition stores, in the Arsenal where large quantities of cordite and other explosives were kept. By his great bravery and his presence of mind, he assisted in extinguishing the conflagration, thus averting serious consequences. For this, he was awarded the Albert Medal.

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[15] A bibliography of Professor Unwin's numerous published works has been prepared and a copy, together with a copy of this memoir, is on file in Engineering Societies Library, 33 West 39th Street, New York, N. Y.

[1] Memoir prepared by Maj. R. N. Aylward, D. S. O., M. C., Lockerley, Romsey, England.

In 1907, Lieut. Battye returned to England to take up a special study of electrical and mechanical work, first with the Manchester Corporation and Messrs, Siemens Brothers, at Woolwich and Stafford, and, afterward, with Sir Alexander B. W. Kennedy, Consulting Engineer, of Westminster, where he worked on various projects, including the hydro-electric plant of the British Aluminum Company in Scotland. After this course, Lieut. Battye returned to India, in 1909, where his services were requisitioned by the Punjab Government to design and take charge of the construction of the new pumping and hydro-electric schemes at Simla, the latter being the first of its kind in the Province.

When the World War broke out in 1914, Captain Battye, then in command of the 21st Field Company of the 3d Sappers and Miners, arrived in France before Christmas, in time for the first Battle of Neuve Chapelle. His inventive mind, backed with tremendous energy, was a great asset to his country and its allies during the war. In the early days, at Givenchy, he introduced a system of electric searchlights to compete with the German star shells; he also devised portable trussed bridges for the infantry to cross the "dikes" which were such a great obstacle to the troops in Flanders. He invented several very effective bombs and grenades and started a factory for the manufacture of them and other "trench stores," the lack of which was so seriously felt in the early days of the war. His earliest bomb was composed of a jam tin loaded with a gun-cotton primer, a proportion of missiles, and a piece of time fuse into the end of which the head of a fuse match was fixed. This proved invaluable at a critical moment in consolidating an important position on the Bethune Front; but a still more useful type of bomb was afterward introduced by Captain Battye and from this the standard type used later in the war was evolved.

In 1915, he was wounded and sent to England. During his convalescence, and before returning to France, he gave a course of lectures and practical instructions to the Canadian troops stationed at Shornecliffe. These lectures were embodied in a pamphlet entitled "The Minor Tactics of Trench Warfare," which was used at other training centers. Before the end of the year, he was again in France, this time as Adjutant, Royal Engineers, to the Lahore Division. A few weeks later he became General Staff Official 3 and, early in 1916, Brigade Major to the 141st Infantry Brigade. In May of the same year he won the Distinguished Service Order for gallantry in reconnoitering and conspicuous bravery at Vimy Ridge. A month later, Major Battye was employed on the General Staff in the 3d Army Corps, and served in this capacity throughout the Battle of the Somme. During this period, he drew up the orders for the first Battle of Cambrai, in which tanks were used for the first time. After this, he originated and organized a new system of forward transportation, in order to expedite delivery of munition and food supplies from the Light Railway limits to the front-line trenches and, at the same time, eliminate, to a great extent, the employment of carrying parties drawn from the fighting units and the consequent casualties in their ranks. Early in 1918 he was made a Brevet Colonel and assisted, as General Staff Officer 1, in the advance to Mons. At the end of hostilities "as a thanks offering for having come through the war safely," Colonel Battye took up work with the American Young Men's Christian Association at its Headquarters in Paris as Liaison Officer between the American and British troops. He always had been a very "live" member of the Young Men's Christian Association and into this task he threw his usual enthusiasm and energy, doing much useful work.

Colonel Battye was on two occasions offered a nomination for the Staff College, but after so many years of destruction caused by the war, he craved for work of construction and decided to revert to the engineering side of his profession where this could be done. At this time, the Punjab Government had applied for his services for hydro-electric work, and here, in this vast agricultural country—a country he loved—his ambition was to see the development of industry without the crowding in smoke-enveloped cities which marks industrial work in most parts of the world. He proposed the formation of a Hydro-Electric Commission, similar to that in operation in Canada and other countries, composed partly of business men from different parts of the Province and partly of Government officials; this plan, however, has not yet been adopted by the Indian Government. He first worked out the designs and estimates for an 80 000-kw project on the Sutlej River with its network of distribution from Delhi in the southeast to Peshawar in the northwest; but, later, a project in the Kangra Valley on the River Uhl, a tributary of the Beas, was selected as being the better proposition. This project has an initial development of 68 000 hp with an ultimate expansion to about 150 000 hp. The head-works, 6 000 ft above sea level, take the water from a turbulent snow-fed mountain stream and direct it through a pressure tunnel nearly 3 miles in length penetrating the Himalayan hills. From the tunnel exit, penstock pipes carry the water to the power-house nearly 2 000 ft below, and the energy, at 132 000 volts, is transmitted 300 miles to the plains, where the distribution system extends over about one-half the Province of the Punjab.

Only a few months before power became available from the Uhl project, Colonel Battye was killed in a motor accident while on a tour of inspection. This tragic ending came when the fulfillment of his aims for the good of the people of the Punjab were about to materialize and in sight of the time when he would have withdrawn from engineering and military duties to retire to family life and consecrate the remainder of his years and his abundant energy to a wider field. He was a serious thinker, both in engineering and social problems, and once convinced of the rightness and soundness of any cause or project he would devote his energy and enthusiasm to see it through. The welfare of his employees was very near his heart, and he did not rest until he had obtained the necessary funds and staff for its organization, materializing in a Welfare Center on the works in charge of a trained official.

Colonel Battye was a deeply religious man, always ready to help others, an active member of the Young Men's Christian Association and a keen Rotarian; at one time he was President of the Rotary Club of Lahore. His interests were many and varied, although he was a man of simple tastes. He was a great soldier, a most enthusiastic worker at anything he undertook, a man of terrific energy both mentally and physically, and, above all, he was always full of good cheer. He was loved by those with whom he worked, both Europeans and Indians, in all ranks, and his passing has left a great blank in many hearts.

He was married on September 30, 1903, to Edith Lilian Cole, of Bristol, England, and is survived by his widow and three sons and two daughters.

Colonel Battye was elected a Member of the American Society of Civil Engineers, on April 25, 1921.

FRANK GEORGE BAUM, M. Am. Soc. C. E.[1]

Died March 14, 1932

Frank George Baum was born at St. Genevieve, Mo., on July 18, 1870, the son of Christian and Caroline (Klein) Baum. His boyhood was spent in Missouri. As a young man he went to California, and, as many Americans have done, worked his way through Stanford University. He received his Bachelor's degree in Electrical Engineering at his graduation in 1898, and, in 1899, the University conferred upon him the degree of Electrical Engineer, after a year as Assistant in Electrical Engineering.

Mr. Baum's first employment thereafter was with the Standard Electric Company of California, then building the Electra Plant on the Mokelumne River. From 1900 to 1903, he was Instructor in Electrical Engineering at Stanford University, but was also employed by the Bay Counties Power Company and others in the solution of the problems arising from the transmission of electric energy over constantly increasing distances and higher voltages.

In 1903, Mr. Baum was made Electrical Engineer, and shortly afterward, General Superintendent, of the recently formed California Gas and Electric Corporation, in which position he continued until 1907. During this period there was installed under his direction hydro-electric power plants of about 40 000 kw, with transmission lines and sub-stations. He then opened an office as Consulting Engineer, but retained his connection with the companies in California that later were merged into the Pacific Gas and Electric Company. As Chief Engineer and General Superintendent from 1912 to 1914, he built the Drum Plant of that Company, of 25 000-kw capacity, with a large masonry dam, canals, pipe lines, transmission lines, and sub-station. At another period, he built a complete power plant for the Cerro de Pasco Mining Company in Peru.

It was due to Mr. Baum's foresight that the Pacific Gas and Electric Company adopted Pit River, in Northern California, as a major source of power. Appreciating the power possibilities of this river, he outlined the general engineering development of the stream. He designed three power plants in that region, of which the largest is Pit River No. 1—a 200-mile transmission line operating at 220 000 volts, and a large receiving station.

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[1] Memoir prepared by J. D. Galloway, M. Am. Soc. C. E.

The prominence gained by his work in California led to engagements on other power plants at home and abroad, notably in France and Germany. The re-organization and development of several smaller systems, together with numerous reports, also occupied his attention during the period of the rapid development of the art of electric transmission of power.

Notwithstanding his eminence as a Constructor, the major elements of Mr. Baum's life work are found in the solution of the problems of long-distance transmission. He covered the period when the art advanced from the small beginnings of the last decade of the Nineteenth Century to the present when electric power dominates modern life.

In May, 1900, appeared his paper, written jointly with the late F. A. C. Perrine, M. Am. Soc. C. E., on "The Use of Aluminum Line Wire and Some Constants for Transmission Lines,"[2] a masterly treatment of the problems of transmission lines and their solution.

On the formation of the California Gas and Electric Corporation in 1902, he was called upon to place in synchronous operation, and to unite into one system, a number of power plants situated on various rivers in the Sierra Nevada Mountains; a problem that was solved successfully. He organized the method of compounding synchronous motor-generator sets by which the potential of the sub-station may be kept nearly normal. Years later, in 1927, he announced the invention of a system of constant voltage transmission with distributed synchronous condensers—a solution of the problem of economical transmission of large amounts of electrical power over distances of several hundred miles. Mr. W. S. Rugg, of the Westinghouse Electric and Manufacturing Company, is quoted as saying that,

> "By means of it the capacity of a given line can be increased 75 per cent. at a cost not exceeding 20 per cent. of the original cost of the line, making possible the economical transmission of power from distant waterfalls and ultimately making possible for sparsely settled outlying districts and farms almost anywhere in the United States, a supply of electric power."

During the period, 1920–1921, Mr. Baum made a study of developing insulators and regulating systems which was outlined in his paper "Voltage Regulation and Insulation for Large-Power Long-Distance Transmission Systems"[3], for which he received the yearly Transmission Prize of the American Institute of Electrical Engineers. His last work was a study of electrical problems at an extensive laboratory which he built near his summer home on a river in Northern California. He was the author of the book "Alternating Current Transformers," and an Atlas of the United States Electric Power Industry.

Mr. Baum was married in 1900 to Mary Elizabeth Dawson who, with three daughters, survives him.

He was a Fellow of the American Institute of Electrical Engineers and a member of the American Society of Mechanical Engineers, and of their Local Sections in California. He was also a member of the Engineers' Club of San Francisco, and of the Lotus Club, New York, N. Y.

Transactions, A. I. E. E., Vol. XVII (1901), p. 345.

Loc. cit., Vol. XL (1921), p. 1017.

In the death of Mr. Baum, the country has lost one of the outstanding men connected with an industry that has conferred such benefits upon humanity that few, if any, surpass it. The manifold applications of electric power to lighten the burdens of mankind are apparent to all, and they stand as witness to the great work of a notable group of men among whom Mr. Baum was not the least.

Mr. Baum was elected an Associate Member of the American Society of Civil Engineers on July 9, 1906, and a Member on October 7, 1914.

JOHN CRICHTON BAXTER, M. Am. Soc. C. E.[1]

Died July 27, 1933

John Crichton Baxter, the son of William Baxter and Mary Louise (Mosier) Baxter, was born of Scotch-English parents in East Orange, N. J., on July 16, 1877.

Mr. Baxter's active career began in June, 1898, as a Private in Company M, 2d United States Volunteer Engineers. After the Spanish-American War, he was, successively, Rodman with the Oregon Railway and Navigation Company; Instrumentman with the Oregon Short Line Railway Company, the Northern Pacific Railway Company, and the Grand Trunk Railway Company; Surveyman with the United States Corps of Engineers; and Computer for the City of Tacoma, Wash. This period from 1898 to 1905 gave him a thorough grounding for field and office engineering, and in February, 1905, he became Resident Engineer of the Alaska Central Railway Company, on construction, and, later, Division Engineer of the same railroad.

In 1907, the Chicago, Milwaukee and St. Paul Railway Company began its western invasion, and Mr. Baxter was drafted as Locating Engineer and, later, as District Engineer on construction. In these years of railroad work, he came in contact with John F. Stevens, Past-President and Hon. M. Am. Soc. C. E., and a close friendship developed between these two men. When the Great Northern and the Northern Pacific Railway Companies jointly decided to build into Central Oregon in 1911, Mr. Baxter was chosen to be the Assistant of Ralph Budd, M. Am. Soc. C. E., on the construction of this 125-mile branch into virgin territory. On the completion of this work, Mr. Baxter became Division Engineer of the Oregon Electric Railway Company, again on construction.

This was Mr. Baxter's last engineering position, as his interest in the contracting field resulted in a connection with the firm of Guthrie-McDougall and Company, Contractors, in March, 1912. As Construction Superintendent for this Company, he was in charge of building the Wenatchee-Pateros

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[1] Memoir prepared by R. F. Hoffmark, M. Am. Soc. C. E.

Branch of the Great Northern Railway, involving about 4 000 000 cu yd of excavation, together with the necessary bridges, station buildings, and tracks. After the completion of this work, he became General Superintendent of the Western Branch of Guthrie-McDougall and Company, and in this capacity he directed the field work of the firm, which included the building of tunnels, dams, and railroads.

In 1915, Mr. Baxter was called to the main office of this firm to direct its extensive operations from St. Paul, Minn. In the meantime the firm of Guthrie-McDougall and Company had become A. Guthrie and Company, Incorporated, and had large stripping contracts on the Mesaba Iron Range, a section of the Chicago Drainage Canal, large dam contracts, and its usual amount of railway work. Under Mr. Baxter's direction this firm became the leading independent contractor in the mine-stripping field. It was due to his keen engineering insight and vision that large 8 and 12-yd shovels, equipped with super-heaters, were designed and used instead of the small standard shovels on the Iron Range work.

In December, 1922, Mr. Baxter was appointed Vice-President of A. Guthrie and Company, Incorporated. In November, 1925, his Company was awarded the contract to build the new 8-mile Cascade Tunnel, of the Great Northern Railway, 90 miles east of Seattle, Wash., with the understanding that this tunnel was to be completed ready for operation, in three years. Mr. Baxter was given full directional powers, and due to his untiring energy and sound judgment, all the world's records for tunneling were broken to complete this bore on schedule time. In order to accomplish this remarkable schedule, new and untried methods of tunneling were evolved, which have since become standard in the driving of long bores. Mr. Baxter's courage and steadfast determination to carry the undertaking to a successful conclusion on an unprecedented schedule, were inspirational to the entire force.

The new methods and equipment developed, as well as the speed with which the work was accomplished, created a great demand for Mr. Baxter as a speaker at various gatherings of engineering societies. Subsequently, he contributed several technical papers on tunneling, which are valuable additions to engineering records on that subject. Among these papers was one published by the Society as part of a Symposium on the "Eight-Mile Cascade Tunnel, Great Northern Railway," entitled "Construction Plans and Methods,"[2] for which Mr. Baxter was awarded the Thomas Fitch Rowland Prize by the Society.

In 1928, the Cerro DePasco Copper Corporation retained A. Guthrie and Company, Incorporated, as Consultants on its tunneling problems in the Andes. In the meantime, Mr. Baxter's Company had decided to open a New York Office, and, consequently, he moved to New York as Vice-President and Contract Manager. From here, he directed the operations in the Andes and handled the general contracting work of the Company.

On August 1, 1930, Mr. Baxter resigned from A. Guthrie and Company, Incorporated, to become a partner in the contracting firm of Carey, Baxter, and

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[2] *Transactions*, Am. Soc. C. E., Vol. 96 (1932) p. 950.

Kennedy. This firm held large foreign contracts, and, due to Mr. Baxter, became active in the stripping operations in the Anthracite District of Eastern Pennsylvania. In February, 1931, the firm was offered a blanket contract for the building of many miles of new railway in Russia, as well as the rehabilitation of the entire Russian Railway System. Mr. Baxter went to Moscow, where he remained for three months, working out details with the Soviet representatives. It was at this time that the Russian financial situation showed weakness, and, consequently, the contract was not executed.

On December 7, 1932, Mr. Baxter became President of Carey, Baxter, and Kennedy. He held this position at the time of his death, which occurred on July 27, 1933, in New York City, from injuries received in a traffic accident.

Mr. Baxter was truly a self-made and self-educated man; without the benefit of a technical education, he was one of America's most brilliant engineers. While his business was exclusively in the Civil Engineering field, his interest in all branches of engineering was so keen that his judgment on all technical problems was invaluable. It can truly be said that his vocation and avocation were the same—Engineering.

The Contracting Industry has lost an inspirational leader, whose ingenuity and sound judgment were grounded in an exceptional knowledge of engineering principles.

He was a member of the American Institute of Mining and Metallurgical Engineers, and also of the Engineers Club, of New York City.

He was married on April 18, 1907, to Mrs. Ollie W. (Winnett) Minnehan, in Josephine County, Oregon. He is survived by his widow, a step-daughter, Gladys, an adopted son, Eugene, and a sister, Lucy M. Baxter, of Portland, Ore.

Mr. Baxter was elected a Member of the American Society of Civil Engineers on July 11, 1927.

WILLIAM MURRAY BLACK, M. Am. Soc. C. E.[1]

Died September 24, 1933

William Murray Black was born at Lancaster, Pa., on December 8, 1855, the son of James and Eliza (Murray) Black. He was graduated from the High School in that City; won his appointment to West Point by competitive examination, while in the third term of his Junior year at Franklin and Marshall College; was graduated from the Military Academy at the age of 21, after heading the Class of 1877 for four years; served in the Corps of Engineers through all grades to Major General and Chief of Engineers; and retired in October, 1919.

Appointed Chief of Engineers in March, 1916, General Black will be recorded in history as the directing head of the Corps of Engineers throughout the World War. He will be remembered also as "the man who cleaned up Havana" and established the Public Works Department of the Republic of Cuba. Among civil engineers he stands as a pioneer in his specialty, river and harbor work and port development, and as a noted administrator of public works. In transportation circles he was a recognized authority on the coordination of railway, waterway, and highway traffic. In the Army he was outstanding as a military engineer whose accurate knowledge of tactical principles and requirements, made the efforts of his own Corps conspicuously effective in supporting the fighting troops. By engineers, business men, public officials, and soldiers alike, he will always be thought of as a cultivated, charming gentleman and a man whose work was characterized by originality of conception and boldness of execution.

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[1] Memoir prepared by R. D. Black, M. Am. Soc. C. E.

The preparatory phase of General Black's service as an officer, commenced with his assignment as an Assistant Instructor in Practical Military Engineering at West Point immediately after his graduation. He next served with the Engineer Battalion and was graduated from the Engineer School of Application at Willets Point, N.Y. His grounding in practical construction work was received on lock and dam work on the Kanawha and Ohio Rivers. This was followed by service as Secretary of the Harbor Commission in Philadelphia, Pa. Another four years were spent at West Point, on duty with Company E of the Engineer Battalion and as Instructor in Practical Military Engineering.

His period of high achievement began when he became District Engineer of the Florida District in 1886. At that time Jacksonville was a struggling town; the Florida East Coast Railway terminated at St. Augustine; Southern Florida was almost unknown. Sensing the possibilities of the State as they were realized by only one other man—Flagler—he persuaded the citizens of Jacksonville to undertake a comprehensive port development at their own expense. This resulted in the adoption by Congress of his project for opening the mouth of the St. John's River, which in turn made Jacksonville a thriving seaport and opened the State to the tremendous growth it has since experienced. His surveys and the projects which he prepared, and in part executed, along the Florida East Coast and at Tampa, Key West, and other points in the State, have had a far-reaching influence in the development which followed the opening of the St. John's River to ocean shipping. Through his work in Florida, General Black initiated the practice of using reinforced concrete in fortification construction, and contributed to Engineering Science some of its earliest exact knowledge on littoral drift of sand, the behavior of bars in tidal estuaries, and the art of handling concrete in sea water.

During the next four years, 1891 to 1895, he served as Instructor of Civil Engineering at the Army Engineer School at Willets Point, N. Y., while commanding Company C of the Battalion of Engineers. During this period, his influence began to spread, not only through the student officers whom he taught, but also through his writings. In 1895, he was called to the Office of the Chief of Engineers where he took charge of the Personnel and Fortification Bureaus. Here, he introduced various reforms and improvements, notably in the procedure for developing coast defense projects, and in submarine mining, and acquired a comprehensive knowledge of the work of the Corps of Engineers which became an important factor in his subsequent career.

In March, 1897, he was appointed Engineer Commissioner of the District of Columbia. As such, he conceived and secured the adoption of the solution to the long studied and difficult grade-crossing problem, which brought about the construction of the tunnel under Capitol Hill and the present Union Station. He established standards of widths for city streets. He authorized electrification of street railways. He secured the harmonious co-operation of the architects of Washington in the design of public buildings, and his planning and street layout have been followed in the great expansion of the city to the northwest.

At the outbreak of the war with Spain, General Black, then a Captain, was made Lieutenant Colonel of Volunteers. After serving as Chief Engineer of the 5th Army Corps, he joined the Field Army at Tampa, Fla., assembled its engineer equipment, and organized and trained a battalion of volunteer engineers. On July 25, 1898, he commanded the first landing of the Army in Puerto Rico, driving the enemy out of the Town of Guanica after a brief skirmish. For the remainder of the war he was Chief Engineer of the forces in Puerto Rico under General Miles. There he nearly lost his life from typhoid fever.

On the occupation of Havana, January 2, 1899, General Black was made Chief Engineer of the Department of Havana on the staff of the late General William Ludlow, U. S. Army, M. Am. Soc. C. E., with one civilian clerk as assistant. The city was in a deplorable state, all municipal service either nonexistent or completely demoralized. Drawing on the personnel of a battalion of the 3d Volunteer Engineers, he organized a new Department of Public Works in three days. This department soon established the sanitary conditions which made it possible for General Gorgas to prevent serious epidemics and eradicate yellow fever.

On the arrival of General Leonard Wood, General Black became Chief Engineer of the Island of Cuba. His code of regulations for the conduct of public works remained in force until incorporated in the laws of the Republic, and was the prototype for codes governing its other major departments. His military survey of the Island became the basis for all subsequent mapping, and before his departure at the end of the occupation, projects had been started for the sewering and paving of Havana and other cities, for wharves to permit the abolition of the system of lightering all freight, in force under the Spaniards, and for the improvement and protection of the ocean front of Havana, destined to become famous as the Malecon Drive.

After five years in the United States, General Black returned to Havana and remained throughout the second occupation (1906–1909), as Adviser to the Department of Public Works of the Provisional Government, taking up the work already begun in the first occupation. He left the Republic with an efficient, smoothly running department and many great projects completed or under way—perhaps the most important being the beautification of the Capital City and the system of arterial highways, since carried to completion throughout the length of the Island.

Between his two tours in Cuba, General Black, while a Major and Lieutenant Colonel, organized and commanded the 3d Battalion of Engineers and the Engineer School of the Army, moving both from Willets Point, N. Y., and establishing them at Washington Barracks, D. C., where he re-organized the courses of instruction at the School, and with the Battalion as a field laboratory, modernized the engineer equipment of the Army. He was also in Panama for a year (1903–04), observing the operations of the French Company and representing his Government during the revolt and separation of Panama from the Republic of Colombia; and for two years (1904–06) he was in charge of fortification construction and river and harbor improvements in Maine, where he developed innovations in building construction which have been broadly adopted in civilian practice.

At the end of the second occupation of Cuba, General Black, then Colonel, was ordered to New York City where he remained until he was appointed Chief of Engineers. While stationed in New York he occupied simultaneously the several positions of District Engineer, Division Engineer, Chief Engineer of the Departments of the Coast and Gulf, Chairman of the New York Harbor Line Board, Chairman of the Intra-Coastal Waterway Board, Senior Member of the Board to raise the battleship *Maine,* and member of the Board of Engineers for Rivers and Harbors of the United States. Among his numerous achievements while in New York, perhaps the most outstanding are: The adoption of the present projects for improving the Harlem and East Rivers; the channel in the North River which made possible the present development of the Jersey City and Hoboken, N. J., water-fronts; the improvement of the Upper Hudson, which has since led to the establishment of Albany, N. Y., as a seaport; the removal of the wreck of the battleship *Maine* from Havana Harbor, by dismantling it within a steel sheet-pile coffer-dam of then novel design; comprehensive tidal studies, which have led to public appreciation and measures for the abatement of the menace of sewage pollution in New York Harbor waters; fixing of harbor lines to protect the fairway of the North and East Rivers against encroachment; and the original plan for the great intra-coastal waterway to provide sheltered navigation from Boston, Mass., to the Gulf of Mexico. In New York, as at his previous stations, he endeared himself to citizens and public bodies by his broad-minded fairness and sound judgment in handling difficult questions, where powerful conflicting interests often had to be reconciled.

The history of the Society's participation in the World War, of necessity centers in the activities of General Black. In March, 1916, when he became Chief of Engineers of the Army, military activities were concentrated on the Mexican Border. Foreseeing the great military effort to come, General Black, with the active co-operation of the Society, canvassed the civil engineers of the country and brought the Engineer Officers' Reserve Corps into being and far on the road to availability for war service. Also, in preparation for an advance into Mexico, he developed the first plan for incorporating large bodies of practical railroad men into the Army. This bore fruit almost immediately after the entry of the United States into the World War in the mobilization and shipment to the Western Front of ten regiments of Railway Engineers, the first troops of the Emergency Forces to reach France after the 1st Division, and, later, to become the backbone of the Railway Transportation Corps of the American Expeditionary Forces.

The months between the declaration of war and General Black's retirement on October 31, 1919, were so crowded with responsibilities and accomplishment as to be bewildering in retrospect and to defy concise description. Under his guidance, the Corps of Engineers was expanded with amazing speed from 254 officers and 2 162 enlisted men to a war strength of 10 761 officers and 297 557 enlisted men; and the Army's vast requirements for engineer equipment and materials were anticipated and met. At the same time, the river and harbor improvements of the country were prosecuted with efficiency, and new and intricate problems in co-ordination of rail and water transportation to meet war requirements, arose and were solved.

In addition to his normal duties as Chief of Engineers, General Black supervised the work of the Director General of Military Railways in the United States and was Chairman of the National Committee on Inland Water Transportation and a member of the Committee on Engineering and Education of the Council of National Defense; he was also a member of the National Research Council. In the spring of 1918 he accompanied the Secretary of War on his inspection of the Army in France, and when General Pershing requested that he be transferred to the American Expeditionary Forces, had the bitter disappointment of learning that his services could not be spared in Washington.

The expansion of the Corps of Engineers from peace to war footing was characterized by the necessity for organizing, training, and equipping a great variety of specialized units. In addition to the combat engineer regiments of the new Divisions and regiments to be used as Corps and Army troops, there were railway construction, operating, and shop units; road building regiments; water supply troops; forestry troops; gas regiments; camouflage units; sound and flash ranging units; and replacements for all of them. Moreover, many highly trained Engineer Officers were required on the Staffs of the Army Corps and Field Armies, at the General Headquarters of the American Expeditionary Forces, and also to direct the multitudinous construction activities of the Service of Supply, from the ports of debarkation to the rail-heads, not to speak of the engineer activities at the ports of embarkation and in the procurement of supplies and equipment at home. Furthermore, engineer personnel was drawn on heavily in the original officering of the Construction Division which built the cantonments in the United States.

The Engineers were not alone in having unprecedented problems of organization and specialization, but under General Black's guidance, these problems were solved with exceptional foresight, speed, and effectiveness, and with a minimum of criticism both during and after the war, at times when criticism and dissatisfaction were rife. The high morale which prevailed throughout the Engineer Service and contributed so largely to its success, may be attributed in no small degree to a policy which General Black conceived and followed in making appointments from civil life. Under this policy, initial rank was conferred in accordance with age and experience. Applicants whose ability was not commensurate with the rank appropriate to their age, simply were not commissioned. The justice of this policy was promptly recognized and kept the Engineers practically free from ill feeling or suspicion of favoritism in matters of rank throughout the war.

Among the enduring monuments to General Black's administration is Fort Humphreys, Virginia. Established during the war as the engineer replacement center to keep about 30 000 men in training, it is now the permanent site of the Army Engineer School, with splendid quarters and facilities adequate for the future training of both officers and enlisted men on either a peace or a wartime footing.

On his retirement from active service, General Black became Consulting Engineer to the Emergency Fleet Corporation. Later, he organized, in Washington, the engineering firm of Black, McKenny, and Stewart, and engaged in consulting practice until about 1929. As the American member of the International Whangpoo Conservancy Commission, he went to China where he prepared a project for improving the Harbor of Shanghai. His firm designed and supervised the construction of the extensive improvement at the mouth of the Magdalena River in the Republic of Colombia.

Throughout his career General Black was frequently called upon to make public addresses. His subjects varied from the education of youth to abstruse problems in engineering. He was a pleasing and accomplished speaker who never failed to hold the interest of his audience or to leave his message impressed upon their minds.

As a writer he was logical, lucid, and forceful. He wrote from a broad vocabulary, with a precise choice of words appropriate to both subject and readers, and a pleasing phrasing and arrangement of subject matter. His "United States Public Works," written in 1893, was the first comprehensive description of the River and Harbor Work of the U. S. Engineer Department ever published. Twice his technical papers won for him the highest recognition of the Society; he was awarded the Thomas Fitch Rowland Prize in 1893 for his paper, entitled "The Improvement of Harbors on the South Atlantic Coast of the United States,"[2] and the Arthur M. Wellington Prize in 1925 for his paper on "Waterway and Railway Equivalents."[3] Always his reports commanded careful attention, from senior officers to the committees of Congress. Many times they brought about the adoption of projects of first magnitude and often they prevented ill-advised expenditure of public funds by their convincing exposition of the fallacies of interested proponents of unworthy schemes.

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[2] *Transactions*, Am. Soc. C. E., Vol. XXIX (1893), p. 223.

[3] *Loc. cit.*, Vol. 88 (1925), p. 538.

Possibly inherited, and certainly developed through his own efforts, General Black possessed marked legal talent. In interpreting and applying the statutes governing the military and civil activities of of the War Department, he demonstrated a knowledge of law and its proper application, often superior to that of professional lawyers of high standing.

In engineering thought, General Black was ahead of his time, and had the courage again and again to put into practice untried methods, when convinced only by his own judgment of their feasibility. This apparent tendency to radicalism was tempered by such clarity of thought and such accurate realization of possibilities and limitations, that his engineering decisions, with the rarest exceptions, eventually proved correct.

As an administrator it was his practice to place heavy responsibility on his assistants, giving them great freedom of action in order to develop and make the most of their initiative. This, coupled with consideration, tact, and firmness, was in no small degree responsible for his success in administering the work of large organizations.

General Black was always a soldier. He believed that theoretical and practical knowledge of the art of war is indispensable to the proper application of engineering in war, however necessary the attainment of practical engineering knowledge may be to the soldier. Accordingly, the civil work of the Corps of Engineers in time of peace was to him primarily a training for its work in war. This idea he strove to instill in his own Corps and the Army at large.

Throughout his life, General Black's superb sense of duty was his guide to the right course of action in every combination of circumstances, and enabled him to carry on courageously and cheerfully through all manner of personal disappointments and discouragement. He was absolutely scrupulous in all matters; he was fair and just in thought and dealing; he had the courage to speak and act in accordance with conviction; and he possessed untiring energy and enthusiasm to the end of his life. He was a leader who created confidence and justified it; he was an inspiration to effort, an example compelling admiration and emulation; within the broadest meaning of the words, he was a loyal citizen and a splendid public servant.

For "Gallant and Distinguished Service" during the war with Spain, General Black was recommended by the Chief of Engineers for the brevet rank of Colonel, and "especially for his professional skill in modifying and improving the sanitary condition of Havana," was recommended for appointment as Brigadier-General of Volunteers. As Chief of Engineers, he was awarded the Distinguished Service Medal for "especially meritorious and conspicuous service in planning and administering the Engineer and Military Railway Services during the War." In 1912, he received the degree of Doctor of Science from Franklin and Marshall College; and, in 1920, the degree of Doctor of Engineering from the Pennsylvania Military College. He became a Fellow of the American Association for the Advancement of Science and was the first President of the Society of American Military Engineers. He was a member of the Protestant Episcopal Church and a Vestryman of St. John's Church, in Washington, D. C.

In 1877, General Black was married to Daisy Peyton Derby, who was the daughter of Captain George Horatio Derby, Topographical Engineers, U. S. Army. She died at St. Augustine, Fla., in 1889. In 1891, he was married to Gertrude Totten Gamble, daughter of Commander William M. Gamble, U. S. Navy.

General Black died in Washington, D. C., on September 24, and was buried at West Point, N. Y., on September 26, 1933. He is survived by his widow and three sons, Roger Derby Black, M. Am. Soc. C. E., Percy Gamble Black, and William Murray Black.

General Black was elected a Member of the American Society of Civil Engineers on June 6, 1888.

FREDERICK KELLOGG BLUE, M. Am. Soc. C. E.[1]

Died May 31, 1933

Frederick Kellogg Blue was born at Trumansburg, N. Y., on May 11, 1866, the son of John and Henrietta (Kellogg) Blue. His early education was received in the public schools, Normal School and College for Teachers, at San Francisco, Calif., and by study and research work at the Mechanics Institute of San Francisco.

In pioneer days, from 1892 to 1899, Mr. Blue was identified with the hydroelectric power development as Engineering Draftsman and Chief Designer with the Pelton Water Wheel Company of San Francisco. He remodeled all the Pelton buckets in accordance with better design for greater efficiency. He also developed many special designs to meet difficult conditions. These designs have since become standard for large wheels of great power and proportionately large jets.

As Chief Draftsman, Superintendent, and Consulting Engineer, for various firms in California, such as the Risdon Iron and Locomotive Works, Union Iron Works, C. C. Moore and Company, San Pablo Quarry Company, Woodruff Construction Company, etc., from 1899 to 1907, Mr. Blue had a varied and successful career. He took a prominent part in the design of steam power plants, mining machinery, balanced electric deep-mine hoisting plants, rock-crushing apparatus, material-handling appliances, power transmission, factory construction, etc.

From 1908 to 1919, he was engaged as Assistant Engineer and Civil Engineer with the Board of Public Works of San Francisco, on water supply, street railways, tunnels, garbage incinerators, etc.

Except for brief engagements as Engineering Draftsman with the San Francisco Offices of Stone and Webster, and as Mechanical Engineer in the San Francisco Office of Ford, Bacon, and Davis, Mr. Blue was employed as Engineering Draftsman with the California Highway Commission from 1920 until his retirement from active engineering practice in 1927.

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[1] Memoir prepared by A. E. Roberts, Cons. Engr., San Francisco, Calif.

Mr. Blue was recognized as one of California's outstanding engineers in advanced applied mathematics, applied science, and research work. He was called upon quite frequently to help solve problems which required deep study and much theoretical knowledge and practical experience.

He was a member of the American Economics Association, and the author of a paper describing "Experiments in Sand-Carrying Capacity of Water Flowing in Small Metal Flumes,"[2] as well as papers on water-wheel design, refuse incinerators, the automatic regulation of public utilities, and theoretical economics, in such technical journals as the *Quarterly Journal of Economics, American Economics Review, Journal of Electricity,* etc.

He was married in San Francisco on December 31, 1903, to Maude Hall, and is survived by his widow and two children, Harold Kellogg and Genevieve Lucile Blue.

Mr. Blue was elected a Member of the American Society of Civil Engineers on November 21, 1921.

JOHN B. BOTT, M. Am. Soc. C. E.[1]

DIED DECEMBER 3, 1933

John B. Bott was born in Westmoreland County, Pennsylvania, on February 14, 1847. His father, John Casper Bott, who was born in Hesse Cassel, Germany, came to America in 1833, and, landing at Baltimore, Md., together with several friends, walked to Pittsburgh, Pa. His mother was Magdalina Veiock, a native of Rhenish Bavaria. Hence, he came of German stock on both sides. Mr. Bott was the oldest of six children, two girls and four boys.

After going through the local public school, he entered Thiel College, at Greenville, Pa., and was graduated therefrom in the Class of 1874, with the degree of Master of Arts.

Mr. Bott's engineering career began prior to his college days, as he entered the employ of the Allegheny Valley Railroad Company, on May 5, 1864, as a Rodman. He remained with this Company until July, 1867, having been promoted through various grades to be Resident Engineer on construction.

He then entered the service of the United States Government, on surveys with the Army Engineers on the Ohio River between Cincinnati, Ohio, and Cairo, Ill. Leaving the Government service in September, 1868, he was engaged on railroad surveys in Jefferson and Armstrong Counties, Pennsylvania, and from May, 1869, to August, 1870, was employed by the Monongahela Valley Railway Company (later the Pittsburgh, Virginia, and Charleston Railway Company). From September, 1870, until August, 1875, his time was spent at Thiel College as a student and Tutor.

Mr. Bott then resumed his engineering career with the Brownsville Railroad Company on surveys and railroad location on Redstone Creek, Pennsylvania, and, as Chief Engineer, had charge of the construction of the Brownsville and New Haven Railroad until April, 1877, when he entered the service of the Ligonier Valley Railroad Company.

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[2] *Engineering and Mining Journal,* 1907.

[1] Memoir prepared by E. W. Clark, M. Am. Soc. C. E.

On January 1, 1878, Mr. Bott went with the Pittsburgh and Lake Erie Railroad Company, and designed all the masonry structures (except the Ohio River Bridge) for the entire line from Pittsburgh, Pa., to Youngstown, Ohio. In May, 1879, he was with the New York, Lake Erie and Western Railroad Company on the Kinzua Viaduct Line, but, in October of the same year, he re-entered the service of the Pittsburgh, Virginia and Charleston Railway Company on railway surveys and location along the Monongahela River.

From April, 1881, to March, 1883, Mr. Bott was employed on the revision of location and the construction of 48 miles of the Shenandoah Valley Railroad, a subsidiary to the Norfolk and Western Railroad. From March, 1883, to March 1887, he was engaged on surveys and railroad locations in Montgomery and Northampton Counties, Pennsylvania, under Mr. G. W. Leuffer, and in Center, Clearfield, and Jefferson Counties, Pennsylvania, under the late C. S. d'Invilliers, M. Am. Soc. C. E. Mr. Bott was then transferred to the Clearfield and Jefferson Railroad (Bell's Gap, Pa.) as Chief Engineer in charge of surveys, location, and construction from Mahaffey to Horatio, Pa.

In July, 1889, he went to Baltimore to take charge of the reconstruction and extension of the Maryland Central Railway, and in connection with this work was placed in charge, as Principal Assistant, of the surveys, location, and construction of the Baltimore Belt Railroad under the late Samuel Rea, Hon. M. Am. Soc. C. E., Chief Engineer. He continued in this position until September, 1896.

Mr. Bott was in general engineering practice until December, 1897, when he entered the service of the Pennsylvania Railroad Company and was located at Brownsville, Pa., on surveys in the Monongahela Valley. He continued in the employ of the Pennsylvania Railroad Company until his automatic retirement, in 1917, at the age of 70. During this period he had charge of numerous surveys and railroad locations, particularly in Western Pennsylvania; the construction of the Ellsworth Branch of the Pennsylvania Railroad; the Monongahela Railroad (under the joint ownership of the Pennsylvania Railroad and New York Central Railroad Companies), from Brownsville to Browns Landing, Pa., together with certain spurs and branches; the Cherry Tree and Dixonville Railroad; the extension of the Ellsworth Branch to Marianna in Greene County, Pennsylvania; and various other branch-line railroads for the development of the coal fields in Western Pennsylvania.

Mr. Bott was always keenly interested in his profession and particularly in bridges and railroad location. His career is typical of the relation of the engineer to the era of railroad development in which he lived.

In politics he was a Progressive Democrat, but was very much interested in the Henry George movement for single tax. He was an enthusiastic gardener and an omniverous reader, and enjoyed walking across country, almost to the end of his life.

On October 9, 1878, he was married to Alice Lorena Georgiana McKee, the daughter of the Rev. David McKee, Professor of Mathematics at Thiel College, Greenville, Pa., and, at one time, President of that institution. He is survived by his widow, two sons, John and Walter, one daughter, Mrs. S. W. Henessy, and four grandchildren.

Mr. Bott was elected a Member of the American Society of Civil Engineers on October 1, 1890.

CLARENCE HENRY BOWMAN, M. Am. Soc. C. E.[1]

Died September 29, 1933

Clarence Henry Bowman was born at Solon, Iowa, on November 30, 1881, the son of Samuel M. and Matilda M. Bowman. He received his early education in the rural schools of Solon, later attending the Iowa City Academy. His technical education was received at the University of Iowa, Iowa City, Iowa, from which he was graduated in 1907 with the degree of Bachelor of Science in Civil Engineering.

His first year's work following graduation was with the Weber Concrete Chimney Company as an Apprentice and, later, with the Ottumwa Bridge Company, of Ottumwa, Iowa, as a lay-out man. Mr. Bowman then entered the field of railway and highway location, construction, and maintenance in the Pacific Northwest, which was to be his life's work.

From 1908 to 1911, he served as Chainman, Rodman, Inspector, and Instrumentman on the location and construction of the Chicago, Milwaukee and St. Paul Railway, in Idaho and Washington. From 1911 to 1913, he was Instrumentman and Acting Resident Engineer on the construction of the Canadian Northern Pacific Railway in British Columbia, Canada. His work there was on heavy railway construction along the Kamloops Lake and Thompson River Sections.

From there, Mr. Bowman went to the Pacific Great Eastern Railway Company, with headquarters at Vancouver, B. C., Canada, for which Company as Resident Engineer he had charge of various location, topographic, hydrographic, and hydro-electric surveys, as well as the construction and maintenance of main-line railroad. During this period he also had charge of the location and construction of terminal facilities, including docks, engine houses, oil storage tanks, pipe lines, and yards at Squamish, B. C., Canada.

With the completion of this construction in 1916, he went with the Chehalis, Cowlitz and Cascade Railway Company as Resident Engineer on location and construction on the Olympic Peninsula in Washington. After a year in that capacity, he was engaged, during 1917, in making hydro-electric surveys in the State of Washington for the Arnold Engineering

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[1] Memoir prepared by Talcott Moore, Dist. Engr., Wyoming Highway Dept., Sheridan, Wyo.

Company, of Chicago, Ill. During 1918, he was engaged in the location and construction of a logging railroad in the spruce forests in Oregon.

In 1919, Mr. Bowman served for a short time as Resident Engineer for the Washington State Highway Department, leaving that position in June of the same year to accept an appointment as District Engineer with the Wyoming State Highway Department, with headquarters at Casper. In this latter capacity, he continuously served for fourteen years until June, 1933, when he was appointed Superintendent and Chief Engineer.

He had suffered from failing health during the year preceding this appointment and was a sick man when he took over this post. With a sudden turn for the worse in his physical condition, Mr. Bowman tendered his resignation in July, 1933. After some time spent in an effort to regain his health, he returned in the fall to his former post as District Engineer at Casper, in which capacity he was serving at the time of his death.

Mr. Bowman was a man of great energy, and it is believed that overwork was a contributing factor in the failure of his health. He was extremely conscientious and drove himself with a zealousness seldom equalled. He possessed an unfailing loyalty to his friends and was ever surrounded by true, steadfast associates. He was active in local affairs, which concerned the welfare of his community, and had served in many capacities in civic organizations. In addition to a highly successful career in the highway and railway field, throughout his life, he typified that which was best in good citizenship.

In his fourteen years of service as District Engineer, Mr. Bowman demonstrated well his abilities as an Engineer and Executive, and his appointment as Executive Head of the Highway Department in Wyoming was in recognition of these able services. The highway system of the State was developed largely during that period, and he played an active part in that development within his District, which covered a large area in Central Wyoming.

He was a pioneer, and took an active part, in the early development of the low-cost oiled-road surface, which type later assumed so important a position in the highway programs of the Far Western States. The oil treatment of gravel roads by the road-mix method was first used in Wyoming in Mr. Bowman's District. Under his direction many types of gravel and sand clay surfaces were successfully treated with different types of road oils produced in Wyoming. Through his work in the development and application of this type of surface, he gained widespread recognition.

He was only 52 years old at the time of his death; comparatively young in years; but he had crowded much activity into those years. Had he been a little less selfish; been willing to consider himself a little more, and the needs of others a little less, the breakdown in his health would not have come so early in life. However, it was not his will that this should be so.

A wide circle of friends mourn his passing and to his family his loss seems irreparable, but he leaves behind a record of accomplishments, a life of service filled with honest unselfish devotion to those whom he served, to be cherished by those who knew him and his work.

On June 24, 1914, he was married to Ada Beatrice Estey, of Seattle, Wash., and is survived by his widow, one son, and one daughter, his aged mother, two brothers, and three sisters.

Mr. Bowman was elected a Member of the American Society of Civil Engineers on August 28, 1922.

MATTHEW JOSEPH BUTLER, M. Am. Soc. C. E. [1]

DIED JUNE 22, 1933

Matthew Joseph Butler was born of Irish parentage, at Deseronto, Ont., Canada, on November 19, 1856, the son of Tobias and Elizabeth (McVey) Butler.

After completing the public school course, he entered University College, Toronto, Ont., Canada. In 1874, on leaving University, he was articled to the engineering firm of Evans and Bolger, at Belleville, Ont. On the completion of his articles in 1878, Mr. Butler entered private practice and, in 1880, had charge of the extension of the Bay of Quinto Railway. In 1882, he was Engineer in charge of the location and construction of the Thousand Islands Railway, and during the following five years he was active in railway location and construction projects throughout Ontario.

In 1887, Mr. Butler entered the service of the Atchison, Topeka and Santa Fé Railway Company in Kansas and Colorado. In 1890 he returned to Canada to become Manager and Part Owner of the wood-pulp mill, at Riviere du Lac, Que., Canada, and during the following five years was identified with a number of railway and industrial projects, including a cement factory, lumber mills, and brick plants. At the same time he held the position of Chief Engineer for the Bay of Quinto Railway during the construction of the Sydenham Branch of that System. In 1895, he built the Oshawa, Ont., Electric Railway.

From 1895 to 1900, Mr. Butler was engaged in general consulting work and, during this period, he devoted himself to the study of law. He was graduated from Kent College of Law, Chicago, Ill., with the degree of Doctor of Laws, and was admitted to the Bar of the State of Illinois in 1897.

From 1900 to 1904, he was in charge of the building of the Hillsborough Bridge at Charlottetown, Prince Edward Island, during which difficult conditions were encountered, necessitating the sinking of pneumatic caissons 100 ft below tide level. The successful accomplishment of this project was always looked upon by Mr. Butler as his most noteworthy professional work, It brought him to the notice of the Canadian Government by which he was engaged as Assistant Chief Engineer of the 1 800-mile section of the Grand

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[1] Memoir prepared by D. H. McDougall, Esq., Stellarton, N. S., Canada.

Trunk Pacific Railway from Moncton, N. B., to Winnipeg, Man., Canada. In addition to the regular duties which this position demanded, Mr. Butler acted as Adviser to the Grand Trunk Pacific Railway Company in drawing up field specifications under which the road was constructed. This position was held from September, 1904, until May, 1905, when Mr. Butler resigned to receive the appointment of Deputy Minister and Chief Engineer of the Federal Department of Railways and Canals of Canada, and also Chairman of the Board of Management of the Canadian Government Railways. In January, 1910, he resigned these positions to become Second Vice-President and General Manager of the Dominion Iron and Steel Company, Dominion Coal Company, and Associated Companies. This position he filled until 1913. During his tenure of office as General Manager, an extensive construction campaign, including important additions to the steel works and mines, was undertaken and completed.

On leaving the Dominion Steel and Coal Companies he became associated with the Sir W. G. Armstrong-Whitworth Company and was appointed Chief Engineer and Superintendent of Construction for its Canadian plant which was begun at Longueuil, Que., in 1913 and completed in July, 1914, just before the outbreak of the World War. Mr. Butler then became Managing Director and built and designed additions to the plant for the manufacture of high-speed crucible steel and installed electric smelting furnaces to supply many of the special steels required during the war period.

On account of ill health, he resigned from this position in 1918, and to the time of his death in 1933, was practically retired from all professional work, except to act as Adviser in special lines of construction work.

In 1880, Mr. Butler was married to Lorette M. J. Shibley, who predeceased him. He is survived by two daughters.

Notwithstanding his very busy and varied career, Mr. Butler yet found time, in 1900, to act on the Royal Commission on Municipal Assessment and Taxation, and, in 1907, to become a Member of the Royal Commission of Forestry. He was a Fellow of the Royal Society of Arts; a member of the Royal Art Society of Canada; and a member of the Institution of Civil Engineers, of London, England, the Engineering Institute of Canada (President in 1914), and the American Society of Mechanical Engineers. He was decorated by His Majesty King Edward VII as Companion of the Order of St. Michael and St. George for distinguished service in Canada. In 1917, he received the degree of Doctor of Laws from St. Francis Xavier College, Antigonish, N. S., Canada.

Mr. Butler's varied professional experience, his wide range of technical reading, and his keen interest in engineering problems, made his advice on all professional subjects particularly valuable. He showed great interest in young members of his own profession and was always glad to assist and encourage them in their work. The sympathy and understanding he showed in such cases endeared him to all engineers who were fortunate enough to be associated with him. He was always an interesting and pleasant companion, a faithful friend, and a true gentleman. His death removes one of the last of the old school of Engineers who played so important a part in the railway and industrial expansion of Canada.

Mr. Butler was elected a Member of the American Society of Civil Engineers on April 1, 1885.

ELGA ROSS CHAMBLIN, M. Am. Soc. C. E.[1]

Died April 28, 1932

Elga Ross Chamblin, the son of D. A. and Annie (Holmes) Chamblin, was born at Mastodon, Miss., on July 11, 1879. He was a grandson, on his mother's side, of Judge Holmes of Pontatoe County, Mississippi, who served in the Mexican War and, as a Major, in the Confederate Army in the Civil War. His paternal grandfather and his father also served in the Confederate Army.

It is not surprising, therefore, that at the outbreak of the Spanish-American War, Elga Ross Chamblin should enlist with the Second Mississippi Volunteers. On his discharge in December, 1898, he enlisted in the 14th United States Infantry, with which he served in the Philippine Islands during the Insurrection there in 1899 and 1900. In 1900, he was sent with Company L of the 14th U. S. Infantry, to represent the United States in the Chinese Relief Expedition during the Boxer Rebellion in China. He participated in the three-day battle before Tientsin in July, 1900, and was wounded at the battle of Yangtsun, on August 6, 1900. After that engagement about fifty of the wounded men, including Mr. Chamblin, forced Chinese prisoners to pull them down the river in a boat to the American Hospital on the coast.

He had attended the public schools and was graduated from the Panola County High School, at Sardis, Miss., in 1896, and after his discharge from the Army, and while engaged as Carpenter, Draftsman, and Estimator for general contractors, Mr. Chamblin took a course (1901 to 1903), in Architecture and Structural Engineering with the International Correspondence Schools, Scranton, Pa.

From March, 1902, to September, 1903, he was employed as Architectural Draftsman with various architectural and contracting firms in St. Louis, Mo. From September, 1903, to July, 1904, he designed and supervised the construction, for the Missouri, Kansas and Texas Railway Company, of the passenger station, freight depot, roundhouse, and turn-table, at Oklahoma City, Okla., as well as other station roundhouses, etc., along 300 miles of that railroad.

In August, 1904, Mr. Chamblin returned to St. Louis, and was employed until December, 1906, as Architectural Draftsman and Assistant Engineer by various firms in that city. From January, 1907, to June, 1911, he was engaged in private practice, in St. Louis, as an Architect and Structural Engineer. As such, he designed and supervised the construction of the Warner-Jenkinson Factory Building, Missouri Lamp and Manufacturing Company Building, Printers' Syndicate Building, Washington Terrace Apartments, etc.

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[1] Memoir compiled from information on file at the Headquarters of the Society.

Mr. Chamblin then entered the employ of C. D. Hill and Company, Architects, of Dallas and Houston, Tex., as Chief Structural Engineer. In this capacity, he was engaged in the design and construction of the Neuces Hotel, at Corpus Christi, Tex., the Sumpter Building, City Hall, Cadillac Hotel, and Edwards Phillips Building, in Dallas; and Drain Hall, Rogers Hotel, in Waxahachie, and the Queen and Isis Theatres, in Houston, as well as other construction work, the total cost of which was more than $2 000 000.

From August, 1913, to March, 1914, Mr. Chamblin was engaged on valuation work for the Dallas Trust Company and other financial institutions; he also acted as Special Adviser on building loans.

Mr. Chamblin, in March, 1914, accepted a position as Consulting Engineer for the Trinity Portland Cement Company, in Dallas. He continued in this capacity until March, 1917, when he was appointed Engineer to the Portland Cement Association in Texas, which position he held until December, 1918. During this time he carried on an independent practice and served as Consulting Structural Engineer to the Department of Architecture of the Agricultural and Mechanical College of Texas, Grubbs' Vocational College, at Arlington, Tex., John Tarleton School, at Stephenville, Tex., etc.

In 1919 and 1920, Mr. Chamblin was a member of the firm of Bartlett and Rainey, Incorporated, Consulting Engineers, with headquarters at San Antonio, Tex. He then returned to Dallas, and from 1921 until his death, was engaged in private practice as an Architect and Industrial and Structural Engineer, making a specialty of reinforced concrete design and construction. His practice was not confined to Dallas, for he designed and supervised the construction of warehouses, factories, garages, and a Jewish Synagogue, at Fort Worth; buildings for the Magnolia Petroleum Company, at Beaumont, Fort Worth, and Amarillo; the Colorado River Bridge, at La Grange; roundhouses, bridges, and culverts for municipal electric railways; the Liberty Cotton Seed Oil Mill, at Texarkana, etc.

Mr. Chamblin was Past Commander of Fitzhugh Lee Camp No. 4 of the Department of Texas, of the United Spanish War Veterans, and was active in the work of that Association. He was a Thirty-second Degree Mason, a member of the Scottish Rite and of the Order of the Eastern Star, of Dallas, and belonged to the Woodmen of the World.

He was also active in civic work, and had served as a Director of the Dallas Welfare Council. He was a member of the Technical Club of Dallas, and was the author of a treatise on "Plain and Reinforced Concrete."

Mr. Chamblin was a Charter Member of the Texas Section of the Society and was active in its affairs. Because of his willingness, he was frequently called upon for Committee work, and discharged these assignments faithfully.

His great interest in engineering activities was well known and appreciated by his fellow engineers in Dallas and throughout the State.

He died at the United States Veterans Hospital, at Memphis, Tenn., on April 28, 1932, after an illness of six weeks. His body was taken to Dallas, and was buried in Grove Hill Cemetery.

He was married at St. Louis, Mo., on June 17, 1905, to Bertha Elizabeth Minderman. He is survived by his mother, Mrs. Annie Lamb, of Sardis, Miss., and two sisters, Mrs. Pearl E. Bost, of Memphis, Tenn., and Mrs. Lillie H. Woods, of Sardis.

Mr. Chamblin was elected an Associate Member of the American Society of Civil Engineers on December 5, 1915, and a Member on April 21, 1920.

JAMES RUSSELL CHAPMAN, M. Am. Soc. C. E.[1]

Died January 11, 1934

James Russell Chapman, the son of Ozias Goodwin and Elizabeth (Russell) Chapman, was born in Dorchester, now Boston, Mass., on October 31, 1851.

He spent his earlier years in New Bedford, Mass., where he attended the Friends' Academy, the family later moving to Beverly, Mass. He attended the Massachusetts Institute of Technology for one and one-half years, and was graduated from the University of Vermont, at Burlington, Vt., in 1873, with the degree of Bachelor of Arts.

Mr. Chapman's engineering experience began as a Rodman on the construction of the Portland and Ogdensburg Railroad, at Wolcott, Vt., during a summer vacation in 1872. After his graduation in 1873, he was Resident Engineer on the construction of the Carolina Central Railway, at Wadesboro, N. C. In 1875, he was Assistant Engineer on the construction of the Massachusetts State Insane Hospital, at Danvers, Mass., and, in 1877, he went to Kansas City, Mo., and re-located the Kansas City and Memphis Railway from Kansas City to the Rich Hill, Mo., Coal Fields.

From 1878 to 1881, Mr. Chapman was engaged as Chief Engineer on the location and construction of railway lines from Denison, Tex., to Gainesville and to Mineola, Tex., subsequently a part of the Missouri, Kansas and Texas Railway, and as Locating Engineer of the Missouri Pacific Railway from Atchison, Kans., to Omaha, Nebr.

In October, 1881, Mr. Chapman became Superintendent of Bridges, Buildings, and Water Service on the Denver and Rio Grande Railway, at Denver, Colo. The road was new and the Bridge and Building Department was kept busy replacing temporary bridges and culverts with permanent structures, erecting repair shops at Division points, and stations at the principal towns on the System. As with all new railways in the West, there was plenty of variety in the work, with cloudbursts, wash-outs, earth and rock slides, snow blockades, etc., requiring immediate action to keep traffic moving and affording ample opportunity for the energy that Mr. Chapman possessed in a remarkable degree.

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[1] Memoir prepared by Woodbury E. Hanson, Esq., Chicago, Ill.

In 1888, he was made General Manager of the Kansas City, Independence and Park Railway, which he rebuilt and operated until 1891. In June of that year, Mr. Chapman went to Grand Rapids, Mich., as Vice-President and General Manager of the Consolidated Street Railway Company, and converted the cable and horse-car lines to electric traction. There were many problems to be met in making these changes. They were solved successfully by Mr. Chapman in this, his first, experience with electric traction. In 1892, he also was in charge of the building of the South Chicago City Railway, which later became a part of the Chicago Surface Lines.

In March, 1894, Mr. Chapman served in Chicago, as Electrical Engineer of the North Chicago Street Railway, West Chicago Street Railway, Lake Street Elevated Railway, Northwestern Elevated Railway, and Union Loop Elevated Railway—the "Yerkes System." The Lake Street Elevated and the Loop Elevated were changed from steam locomotive to electric traction, the Northwestern Elevated was built, the North and West Side Cable and Horse Car Lines were converted to electric traction, surface lines were extended, and a power house was erected. This was a period of great activity in Chicago traction affairs, and Mr. Chapman spent seven busy years in doing his part toward giving the city its traction system.

In March, 1901, he went to London, England, as Chief Engineer and General Manager of the Underground Electric Railways Company of London, Limited, comprising: The Metropolitan District Railway (56.07 miles); the Baker Street and Waterloo Railway (10.41 miles); the Great Northern, Piccadilly and Brompton Railway (8.02 miles); the Charing Cross, Euston and Hempstead Railway (4.45 miles); and The Lots Road Generating Station. The Metropolitan District Railway was the original Underground Railway. The first section was opened for traffic with steam locomotives in 1868. About one-fourth the mileage is of shallow "cut-and-cover" construction. The other three lines are "tubes" under London streets, with a depth ranging from 30 to 185 ft, and 170 elevators were required for the Tube Stations. The Lots Road Generating Station is built on the River Thames, in the Borough of Chelsea. This site was chosen because coal could be delivered by barge or by rail, and because a water supply could be obtained for the condensers.

The direct-current system was adopted for the Underground Lines. Other railway lines having "running powers" over a part of the District Railway proposed a three-phase system, and arbitration proceedings were necessary to decide the question. The decision was in favor of the direct-current system. Two insulated conductor rails—a positive and a negative—are used, to each track on both the District Railway and in the Tubes. Electro-pneumatic signals were installed and the change from the hand-operative signals on the District Railway, together with the electric traction, increased the maximum capacity of the line from 18 to 40 trains per hour.

In addition to new signals, concrete platforms, new lighting, telephones, a time system and train indicators were installed on the District Railway, and electric traction was put in operation in 1905. There was some complaint at first that the cars were too well lighted, too hot, and the center doors too confusing, but the change from the ill-lighted compartment carriages and the smoke and dirt of the locomotives was greatly appreciated by the patrons of the road.

The electrification of the "tubes" was a much simpler matter than that of the District Railway, where it had to be carried out during traffic, or in the few hours at night when no trains were running. The Baker Street and Waterloo Railway was opened for traffic in 1905, the Great Northern, Piccadilly and Brompton, in 1906, and the Charing Cross, Euston and Hempstead, in 1907.

The Lots Road Generating Station is a monument to Mr. Chapman's ability. It was the first large steam turbine power house built and is still operating efficiently and economically after twenty-nine years in service.

In 1910 Mr. Chapman resigned his position with the Underground Electric Railways System and returned to Santa Barbara, Calif. He served as President of the Board of Water Commissioners of Santa Barbara and directed the construction of the Gibraltar Dam on the Santa Ynez River, the source of the water supply for Santa Barbara.

Mr. Chapman was a tireless worker with an extraordinary grasp of detail. His men responded to his demand for the best they could give. He always had their loyalty and respect. Furthermore, he possessed the trust and confidence of those whom he served to a high degree, and was allowed to carry out his plans and do his work as he thought best. This was perhaps because he worked out his own problems and never troubled his employers with details.

The esteem in which he was held was expressed by Sir Edgar Speyer, Chairman of the Underground Company, at a testimonial dinner given Mr. Chapman when he was leaving London. This dinner was attended by the Directors of the Underground Company, and the Board of Trade Officials and Engineers with whom he had been associated during his stay in London. Sir Edgar said:

> "He has done a great and wonderful work for London that will live after him as a monument to his remarkable capacity, foresight, and skill, and he has done it modestly, quietly, thoroughly, and well. It is a remarkable achievement, and the Underground Company is proud of it as they are proud of him, and I can assure him that the second monument he has erected is as fine and will be as lasting. It is erected in our hearts from very solid materials: Gratitude, Respect, and Affection."

He was married, in 1879, to Antoinette Hagar, of Salem, Mass. Mrs. Chapman and a daughter, Mrs. H. Paul Ahrnke, of Berkeley, Calif., survive him.

Mr. Chapman was elected a Member of the American Society of Civil Engineers on March 6, 1901.

CLEMENT EDWARDS CHASE, M. Am. Soc. C. E.[1]

Died September 18, 1933

Clement Edwards Chase was born in Omaha, Nebr., on July 26, 1888, the son of Clement and Lula E. Chase. He was graduated from the Omaha High School in 1906, and from Cornell University, Ithaca, N. Y., in 1910, with the degree of Civil Engineer.

Mr. Chase began the active practice of engineering during his school vacations, working, in the summer of 1907, as a Time-Keeper on the Wright Creek Dam, at Cameron Pass, Colorado. In the summers of 1908 and 1909, he was employed as Rodman, and Concrete and Steel Inspector, on the erection of the McKinley Bridge over the Mississippi River, at St. Louis, Mo., this being his first engagement with Ralph Modjeski, M. Am. Soc. C. E.

In 1910 and 1911, Mr. Chase was Assistant Engineer with the Oregon Trunk Railway Company, in immediate charge of the Field Party, supervising the construction of substructures of bridges; and, afterward, Inspector of Erection of the Crooked River Arch, in Oregon, all this work having been designed by Mr. Modjeski. He was then transferred to the Cherry Street Bridge, in Toledo, Ohio, where he acted as Assistant Engineer on construction. After completing this work he was engaged to represent the firm of Modjeski and Angier as Engineer, in charge of mill and shop inspection, in the Pittsburgh, Pa., Office. This was the beginning of a continuous life-long association with Mr. Modjeski, in various capacities. During this time, Mr. Chase assisted with the field inspection of bridges of the Pennsylvania Lines West of Pittsburgh subsequent to the flood of 1913, and also in the preparation of a report for the Pennsylvania Railroad Company on the raising of its bridges in Pittsburgh.

In 1917 and 1918, Mr. Chase was Resident Engineer for the designer, in charge of the construction of the Hudson River Bridge, at Poughkeepsie, N. Y., for the Central New England Railway Company, which structure was remodeled under traffic. In 1918, in addition to his regular work in the New York Office, he looked after the personal business of F. M. Masters, M. Am. Soc. C. E., during Mr. Masters' service in the Army, and had charge of the supervision of mill, shop, and field inspection of the Ohio River Bridge, of the Pennsylvania Railroad Company, at Louisville, Ky., and the Pennsylvania Railroad Bridge, at Rochester, Pa.

From 1919 to 1920, Mr. Chase acted as Assistant Engineer in charge of the New York and Pittsburgh Offices of his employer, during which time he reviewed and checked the design for the three-span, continuous-truss bridge of the Cincinnati and Southern Railway Company, over the Ohio River, at Cincinnati, Ohio. In 1920, he accepted the position of Principal Assistant Engineer to the Delaware River Joint Commission, under the Chief Engineer of the Commission, and served during the work on the preliminary report, design, and construction of the suspension bridge over the Delaware River between Philadelphia, Pa., and Camden, N. J., having charge of the preparation of contracts and specifications, general supervision of design, mill and shop inspection, special research, tests, and investigations. Upon the completion of this structure, he joined with Mr. Modjeski and Mr. Masters in forming the firm of Modjeski, Masters, and Chase. He had active charge of the design and construction of the Tacony-Palmyra Bridge, over the Delaware River and, at the same time, served as Consultant on the design and construction of the Ambassador Bridge across the Detroit River, at Detroit, Mich., and on the two Henry Avenue Bridges for the City of Philadelphia.

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[1] Memoir prepared by a Committee of the Philadelphia Section consisting of Henry J. Sherman, and Frank M. Masters, and Russell G. Cone, Members, Am. Soc. C. E.

In 1930, the firm was incorporated, and Mr. Chase was active in its work in connection with the preparation of numerous reports and investigations of existing structures; reports on the construction of bridges for various commissions, such as the Mississippi River Bridge, at New Orleans La., for the Public Belt Railway Commission; report on the high-speed lines for the Delaware River Joint Commission; the Mississippi River Bridge, at Davenport, Iowa; the report to the State of New York on the construction of the St. Lawrence River bridges; and the reports on the Klingle Valley Bridge, on the line of Connecticut Avenue, and the Calvert Street Bridge, over Rock Creek Park, for the District of Columbia.

His sudden death cut short a remarkable career. Although only 45 years of age, Mr. Chase had been in charge of the design and construction of many large and monumental structures, among which were the Delaware River Bridge, at Philadelphia, and the Ambassador Bridge, at Detroit, both of which, at the time, were the longest suspension bridges in the world. Those who knew him believed that he was at the threshold of a great career. A rapid but sound thinker, his ideas on bridge design and construction were always in advance of the profession in general, and the art has sustained a great loss in his passing. Mr. Chase was deeply interested in his profession from earliest youth. He was prominent in the affairs of the various technical organizations to which he belonged, and his judgment and opinions were highly regarded by those whose privilege it was to have been associated with him in such matters.

Mr. Chase's standing in the profession was due, not only to his fine personality, but also to the fact that he was a tireless worker and had the advantage of an unusually keen mind. He would pursue the details of his work to the very end, no matter whether the path led through the intricacies of the computations and the drawings of the design office, to the caisson's bottom, or to the highest girder of one of his structures. It was the latter circumstance that led to his sudden and untimely death. While examining a connection on one of the stringers for the high-speed line of the Delaware River Bridge, he fell from the structure to the ground, more than 100 ft below, and was instantly killed.

Mr. Chase was a member of the American Society for Testing Materials, Cornell University Clubs, of New York and Philadelphia, the Engineers'

Clubs, of Philadelphia and New York, and the St. David's Golf Club, of Philadelphia. From Cornell University he received the Fuertes Medal, and, in 1917, he was awarded the Collingwood Prize for Juniors by the Society for his paper[2] on "The Cherry Street Bridge, Toledo, Ohio." He was a member of the Sphinx Head and the Phi Gamma Delta Fraternity. Another honor which came to him was an appointment as Brackett Lecturer, at Princeton University, in 1925.

He was married on June 10, 1914, to Helen Kelsey, of Toledo, Ohio, and to them were born three children, Clement, Alice, and Sherret, all of whom survive him. A successful life was rounded out by the happiness of his home life. He had the same enthusiasm at the fireside that he carried to the design and conference rooms. Mr. Chase enjoyed a wide circle of friends and admirers all over the United States and in all walks of life. Numbered among them were many of those high in the country's esteem, all of whom deeply mourn the abrupt and tragic conclusion of his brilliant career and the loss of their beloved friend.

Mr. Chase was elected a Junior of the American Society of Civil Engineers on October 1, 1912; an Associate Member, on April 19, 1920; and a Member on October 21, 1924. He was a member of the Philadelphia Section of the Society, of which he was Vice-President in 1928–29.

OSCAR CLAUSSEN, M. Am. Soc. C. E.[1]

Died October 26, 1932

Oscar Claussen, the son of Paul and Caroline Marie (Creutzfeldt) Claussen, was born at Terre Haute, Ind., on December 2, 1860. His parents emigrated to the United States in 1852, coming to Terre Haute where his father established and conducted a wholesale grocery store, which business was managed very successfully until ill health forced the father to retire in 1869, at which time the family returned to Germany.

Oscar Claussen was nine years old at the time. Before going back to Germany with his parents the boy had attended the grade schools in Terre Haute; his grade and High School education was completed in Wiesbaden, Germany. Later in his life, the friendships he made during his school days in Terre Haute and his love for his native place were to play a major rôle in his decision to return to his friends of boyhood days and his native land.

In 1878, Mr. Claussen entered the Federal Polytechnic Institute, of Zurich, Switzerland, from which he was graduated with honors in 1882. Under the well-known engineer, Professor Dr. E. Culman, he studied bridge building and "graphik statik," of which Dr. Culman was the founder. While at Zurich, Mr. Claussen was a student of the celebrated Professor Tetmeyer who was the

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[2] *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 744.

[1] Memoir prepared by James E. Carroll, M. Am. Soc. C. E.

Director and Founder of the "Eidgenossischen Testigkeits Prüfungs Anstalt" (Institute for Testing Materials), and the directing genius and head of the Swiss Government Laboratory for Testing Building Materials. On receiving his diploma Mr. Claussen was appointed as Professor Tetmeyer's Assistant, which appointment was not only a recognition of Oscar Claussen's scholarly attainments, but also the esteem in which he was held, and presaged a useful and honorable career. As Professor Tetmeyer's Assistant Mr. Claussen was particularly occupied with testing specimens of Swiss lumber suitable for structural purposes. The data from these tests were published in book form, a copy of which was issued by the Government at the National Swiss Exhibition of 1883.

Mr. Claussen spent fourteen years in Germany and Switzerland at this time, years of exceptional value in education and professional training. Although his future there was assured, the strong wish to return to the land of his birth remained, and he left Switzerland for America. Confidence in himself and high courage prompted the move, opposed as it was by the arguments and persuasions of his former professors and classmates. The wisdom of his choice is supported by his distinguished success in accomplishments and in the positions he held, each a step higher in the line of human endeavor and an ever larger field for public service. Therefore, in 1883, Mr. Claussen resigned his position at Zurich and returned to his native land. Soon after he arrived in the United States, he entered the employ of the Keystone Bridge Company, at Pittsburgh, Pa., with which Company he was engaged until the spring of 1885, when he left Pittsburgh and went to St. Paul, Minn. Soon after his arrival he was appointed Bridge Engineer of the St. Paul and Northern Pacific Railway Company (now part of the Northern Pacific Railroad System), by the Chief Engineer, J. W. Kendrick, M. Am. Soc. C. E.

In 1886, Mr. Claussen made a trip to Europe and spent six months on the Continent, during which time he visited and studied outstanding engineering projects, particularly bridges, river and harbor terminals, and the fine examples of landscape engineering in France, Germany, and Switzerland. Being a proficient linguist, his visit was very enjoyable and profitable. On his return in 1887, Mr. Claussen became Engineer in Charge of Bridges and Water Supply for the Chicago, Great Western Railway Company, which employ he left to enter the service of the City of St. Paul, where he remained until 1890.

From 1890 to 1893, Mr. Claussen was Engineer for the Northwest General Electric Company of St. Paul, during which time he was in responsible charge of all water power development and electric railway central station power plants. During these years he also had charge of the construction of the St. Cloud Street Railway, the St. Paul and White Bear Electric Railway Line, and the Ashland, Wis., Street Railway.

From 1893 to 1899 he was engaged in practice as a Consulting and Contracting Engineer. During these years he designed and built numerous electric light plants, water-works systems, and also sewerage systems. In 1894, Mr. Claussen designed and had charge of the bridge and spiral approach at

Hastings, Minn. This approach is the only one of its kind in the world, and its building is a fine example of the ability and resourcefulness of Mr. Claussen, who when called upon by the City of Hastings, solved the difficult bridge-approach problem. This approach was designed primarily for horse-drawn vehicles and, therefore, has outlived its original complete usefulness. The bridge continues to serve its purpose as a busy river crossing for modern vehicular travel of many kinds and will continue to do so until another scientific engineer solves to-morrow's problem for the growing traffic to and from the City of Hastings.

From 1899 to 1902, Mr. Claussen was Commissioner of Public Works and Chief Engineer of St. Paul. He held this position, again, from 1911 to 1922, when he resigned. During the years between his first and last term as Chief Engineer of St. Paul, he was engaged in the Consulting Engineering business. Many of the younger cities of the great Northwest have outstanding examples of his work in the design and construction of municipal lighting plants and water and sewerage systems.

In 1922, he organized the firm of Claussen and Carroll, Engineers, and was Senior Member of the firm, which was engaged in general engineering practice, until 1927. From 1927 to 1932 he was Vice-Chairman of the Metropolitan Drainage Commission, an activity which greatly appealed to his desire for public service. Of his time and talents, he gave generously and without remuneration. This was his culminating and absorbing interest and one which gave him the greatest satisfaction.

Mr. Claussen was a member of the old "Board of Electricity of Minnesota" and was its Treasurer from January, 1901, to 1907. He served as President of the Engineers Society of St. Paul (1906-1907). He was also a member of the old Commercial Club and its successor, the St. Paul Athletic Club, and a member of Lodge No. 59, Benevolent and Protective Order of Elks.

Mr. Claussen was keenly alert and active, with a wide range of interests. He was a man of untiring industry, and scrupulous in his observance of business ethics. He was averse to publicity, being of a retiring nature, quiet in his tastes, and fond of reading, particularly history and economics. He was a great "home body" and took an absorbing interest in his family and the home where he spent many happy hours.

A former college classmate in Zurich, Switzerland, keenly affected by the news of his death, wrote: "Oscar Claussen was very popular among all his fellow students, friendly, courteous, in all a noble character."

In closing this brief tribute to an eminent engineer and citizen the following excerpts from the leading papers of his home city, St. Paul, Minn., show the esteem in which he was held. In an editorial in *The Pioneer Press*, of January 26, 1903, it is stated:

> "During his incumbency Mr. Claussen has given constant proof of watchfulness, of progressiveness, and of courage.
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> "Most of the things that Mr. Claussen has accomplished and most of the things which he has attempted to accomplish have been undertaken in the face of the opposition of interests, which, as events have shown, exert more than their share of influence on the Council."

From the *St. Paul Dispatch* is taken the following:

"St. Paul never had a more worthy citizen, or a more public-spirited man than Oscar Claussen. His sudden death will be a shock to thousands. I had known him intimately for a quarter of a century and always found him faithful in his friendship and inspiring in the quality of serenity and sureness which marked his thinking. He was not the parading kind. He was quiet, and unostentatious, but he had a deep seated pride in his work and in his professional standing. I shall always be glad that it fell to my lot to make possible one of his last ambitions. His splendid fidelity to the public interest during his years as a city official will not soon be forgotten. He was fearless, independent, honest, human, which is enough praise for any of the sons of the earth."

Mr. Claussen was married in 1891 to Martha Willius, the daughter of Ferdinand and Clara Holterhoff Willius, both families being prominent in the financial and social affairs of St. Paul. Four children blessed this marriage. His widow and three daughters, Mrs. E. T. Herrmann, Clara, and Frieda Claussen, survive him. One son, Norman F., who saw service in France during the World War as a Captain in the 15th Field Artillery, died in service on October 27, 1918.

The loss of Mr. Claussen's only son was a blow from which he never fully recovered. The son had shown good business ability in the management of the electric light plants at Lisbon, N. Dak., and Milbank, S. Dak. He was a credit to his parents, a brave soldier, a good executive, a kindly and lovable young man who would have become a prominent citizen.

Mr. Claussen was elected a Member of the American Society of Civil Engineers on October 2, 1901.

ALBERT NEUMANN CONNETT, M. Am. Soc. C. E.[1]

DIED JANUARY 1, 1933

Albert Neumann Connett, the son of Eugene V. and Sophia (Rutan) Connett, was born at Bethel, Conn., on July 12, 1859. He was a descendant of James Connett who came from Devonshire, England, and settled in Charlestown, Mass., late in 1600, and who, in 1702, became one of the pioneers of Woodbridge, N. J. James Connett was married to Mehitable Gardner, and of the sixth generation was Daniel Brecount and Mary B. Clarkson, the grandparents of Albert Neumann Connett.

Mr. Connett received his academic education at the Riverview, N. Y., Military School and Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1880 with the degree of Civil Engineer.

From September, 1880, to July, 1881, Mr. Connett was engaged as an Assistant to the late William E. Worthen, Past-President and Hon. M. Am. Soc. C. E., and as a Civilian Engineer with the United States Department of Rivers and Harbors, on the improvement of the Mississippi River, at St. Charles, Mo. Then, for a period of about three months, he served as Resident

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[1] Memoir compiled from information on file at the Headquarters of the Society.

Engineer in charge of track construction for the Toledo, Cincinnati and St. Louis Railroad Company, after which he accepted a position as Assistant Engineer with the Union Pacific Railway Company on the Oregon Short Line Railroad. In February, 1882, Mr. Connett returned to the employ of Mr. Worthen as Assistant Engineer and Principal Assistant Engineer, respectively, and as such was in charge of the construction of water-works at East New York, N. J.

Mr. Connett gave up the practice of his profession in March, 1884, and until April, 1888, was engaged in ranching in Ellsworth County, Kansas.

After his return to the practice of engineering, he was employed until September, 1888, with the firm of Knight and Bontecou, Consulting Engineers, on the laying of cables for street railways in Kansas City, Mo. From that time until May, 1889, as Assistant Engineer with the Bentley-Knight Company, of Boston, Mass., he was in charge of the construction of conduit roads in Boston, and New York, N. Y. He returned to the employ of the late Daniel Bontecou, M. Am. Soc. C. E., as his Assistant on the construction of a cable road for the Washington and Georgetown Railroad Company, in Washington, D. C., and from March, 1890, to 1894, he served as Principal Assistant in Charge of Construction and Chief Engineer of the Baltimore City Railway Company, of Baltimore, Md. From 1894 to 1896, he was Chief Engineer with the Metropolitan Street Railway Company, at Washington, D. C.

It was about this time, during the Nineties, that a change was being made from horse-drawn street cars to lines run by electricity. While Mr. Connett was Chief Engineer of the Metropolitan Street Railway Company at Washington, Congress ordered the electrification of the street railways of the city. This change was to be made within a year. Objections were made to overhead wires and trolleys and the storage battery system proved a failure. Under the stress of this predicament, Mr. Connett designed the first electrical conduit system for street railways ever put to practical service. The installation was a complete success in Washington and was made before the time limit set by Congress. The operation of the Ninth Street Line on August 1, 1895, marked an important advance in the use of electricity in transportation. At first, the conduits were placed underground between the rails, but, later, Mr. Connett invented a side slot in one of the rails and thereby laid the foundations for the present third-rail system.

On account of his inventions, Mr. Connett was selected in 1896 to be Chief Engineer of the Cie Francaise Thomson-Houston, of Paris, France, the French affiliate of the General Electric Company. As such, until 1900, he had charge of the construction of a number of electric railway systems in France.

From 1900 until 1922, when he retired from active work, Mr. Connett was Chief Engineer and Managing Director of J. G. White and Company, Limited, of London, England. During this time he directed the development of electric enterprises in all parts of the world. The street railway systems in Bombay, India, Kalgore, Australia, Auckland, New Zealand, and Para and Manãos, Brazil, were electrified under his supervision; the street railway line at

La Croze, Argentine Republic, was designed and constructed by him; electric plants at Caracas, Venezuela, and Merida, Mexico, were rebuilt and operated under his direction; and he constructed the high-speed line, entirely on piles, from Amsterdam to Scheveningen, Holland, as well as the electric light and railway power plants at Pernambuco, Brazil, and at Santa Fé, Cordoba, and Parana in the Argentine Republic.

During the World War, Mr. Connett joined the Belgium Relief Commission of which Herbert Hoover, Hon. M. Am. Soc. C. E., was head. Mr. Connett organized the work of the Commission, and for several months acted as Director General in Belgium. Through his efforts the relief work was extended into parts of France held by the German armies. He was made an Officer of the Order of Leopold by the Belgian Government for his work with the Commission.

He was Director of many light and power companies, and a member of the Institution of Civil Engineers of Great Britain. He was also a member of the Reform Club of London, the Pilgrims Society, the Union League and University Clubs of New York, and of the Delta Phi Fraternity.

Mr. Connett died at his home in New York City at the age of seventy-three years. In a memoir prepared for the American Historical Society, it is stated that:

> "He had, in a sense, completed a career of extraordinary usefulness to the generation of which he was a part, and in all corners of the world. His life, nurtured in difficulties and noteworthy for its steadfastness of purpose, its energy of execution, its sterling worth, a life singularly rich in experience and beautiful in its simplicity and symmetry, may well serve as an example and inspiration to the youth that must build on foundations that he laid."

He was married on June 3, 1885, to Theodora, daughter of Francis Speir, of South Orange, N. J., and is survived by his widow and four children, Albert Neumann Connett, Jr., of Paris, France; Francis S. and Thomas O. Connett, of New York City; and Harold Connett, of Philadelphia, Pa., and five grandchildren.

Mr. Connett was elected a Junior of the American Society of Civil Engineers on June 6, 1883, and a Member on October 1, 1890.

ABRAHAM FAIRBANKS DOREMUS, M. Am. Soc. C. E.[1]

DIED MAY 10, 1933

Abraham Fairbanks Doremus, the son of Dr. Henry I. and Harriet (Fairbanks) Doremus, was born in Salt Lake City, Utah, on May 24, 1849. On his father's side he was descended from Cornelius Doremus, of Middleburg, Holland, who settled in Paterson, N. J., about 1680. On his mother's side he was descended from Jonathan Fairbanks, who came to America in 1633 and settled at Dedham, Mass. As a child he studied physics and mathematics under his father's direction, later attended the public schools of Salt Lake City, and the University of Deseret in the same city.

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[1] Memoir prepared by a Committee of the Utah Section consisting of George M. Bacon, Richard R. Lyman, Richard A. Hart, and A. Z. Richards, Members, Am. Soc. C. E.

At the age of sixteen Mr. Doremus started his engineering career as Rodman and Chainman on the location of the Union Pacific Railroad, the construction of which was being rapidly pushed toward Ogden, Utah. In 1869, he was Assistant Engineer on location and construction of the Utah Central Railway, 37 miles of road connecting Salt Lake City with Ogden. Next came two years as Assistant Chief Engineer on the location and construction of 75 miles of the Utah Northern Railway, which extended northward from Ogden.

In 1873, he turned his attention for a short period from railroading to take charge of the construction of a canal to furnish power for the reduction of lead and silver ores. Then followed ten more years in charge of location and construction of Utah railroads, interrupted in 1877 for a year while he was First Deputy United States Mineral Surveyor for Utah and then Special Agent for the Department of the Interior, charged with the examination of Government Land Surveys.

With the substantial completion of railroad construction in Utah, Mr. Doremus' connection shifted to maintenance, and from 1884 to 1888, he was in charge of maintenance of way, bridges, buildings, water supply, etc., for the Utah Central Railway Company. During 1889, he was Division Engineer on the Utah Division of the Union Pacific Railroad, covering 400 miles.

By this time his professional experience and standing were such as to draw him out of any special field and, in 1890, he became City Engineer of Salt Lake City, holding this office for two years. During his tenure the City built the Parley's Canyon Conduit, the first large-scale step in providing the city with a modern and adequate water supply. Then followed his appointment as State Engineer of Utah.

The office of State Engineer in States of the arid West has as its chief function the administration of water laws and the supervision of the distribution of the waters of the State. In spite of the fact that Utah was the principal pioneer in irrigation, it had lagged behind some of the other arid States in keeping its water laws abreast of the latest practice. During his term of office, Mr. Doremus was largely instrumental in thoroughly modernizing the law covering the functions of his office and, in particular, laid the foundations on which is built the present structure of laws governing the scientific gathering of data which permit of clear and explicit definition of water rights in Utah.

At the expiration of his term of office in 1905, he entered the career of Consulting Engineer, which was brought to a close only by his death. Space does not permit of a catalog of all his activities. Mention should be made of his acting as Consulting Engineer for Salt Lake City during the construction of the Big Cottonwood Conduit.

A partial return to public office took place in 1921 when he was appointed a member of the Utah Water Storage Commission. This newly created body had as its function the correlating of private and public irrigation construction and long-time planning. Mr. Doremus was at once given the prominent part of gathering information and data on which the principal work of the Commission has ever since been based. At the expiration of each term of office, he was re-appointed and, for the past eight years, since 1925, he had served as one of the two Vice-Chairmen of the Commission. An abstract of a Resolution offered by the Commission to the memory of Mr. Doremus, is, as follows:

"Commissioner Abraham Fairbanks Doremus having been born in Salt Lake City less than two years after the original Pioneers came into the Valley has seen with his own eyes much of the industrial and other developments of his native State. With his efficient hands and keen intellect he has made many effective and important contributions to its physical and social development. Born on the frontier, cradled in the Great American Desert, blessed with great mental power, unusual foresight, sterling character and genuine worth, this rugged pioneer, imbued with the spirit of genuine culture, has from the days of his boyhood been an active force in many of the outstanding civic and business developments not only of his native State but of the Intermountain West."

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"It was a fitting climax to his lifetime of efficient and accurate devotion to engineering problems, especially those connected with water, that he was given an opportunity to serve as a member of the Utah Water Storage Commission from the time of its organization to the day of his death. He put into the work of the Commission very much of his time and very much of his thought in an effort to assist in putting to beneficial use all the remnants of the comparatively meager water supply which exists in the State of Utah. Members of the Utah Water Storage Commission regard it as an honor to have been associated with this dignified and outstanding leader for so many years."

With his reaching an assured professional reputation, Mr. Doremus took a greater and greater interest in the younger engineer. His uniform kindliness in his contacts with others did not preclude his speaking his mind when definite conviction led him to differ with his associates. Firm in his opinions, he never refused to give the other side a hearing. In all relations, his unfailing courtesy could be counted on by every one.

On November 15, 1869, he was married to Paulina Richards, of one of the leading Utah pioneer families. He is survived by Mrs. Doremus, a daughter, Mrs. Hattie D. Hagman, and two sons, Henry R. and Cornelius R. Doremus.

Mr. Doremus was elected a Member of the American Society of Civil Engineers on October 4, 1893.

HERBERT FRANKLIN DUNHAM, M. Am. Soc. C. E.[1]

DIED DECEMBER 1, 1933

Herbert Franklin Dunham was born on a farm at Savoy, Berkshire County, Mass., on January 27, 1850, the son of Alvin Franklin Dunham and Sarah

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[1] Memoir prepared by D. C. Henny, M. Am. Soc. C. E.

(Kennie) Dunham. When the boy was ten years of age, the farm was sold and the family moved to Windsor, Mass., where they lived until they moved to West Chesterfield, Mass., two years later. Mr. Dunham retained possession of the home at West Chesterfield until his death.

He began his professional career at the age of 21 in Springfield, Mass., and, later, worked in engineering offices in Worcester and Boston, Mass. The sphere of Mr. Dunham's professional activities gradually extended westward. His work encompassed railroad, water-works, electric light plant, and bridge construction throughout the Middle Western States. For many years after 1882, he maintained offices in Cleveland, Ohio, in partnership with the late Mr. C. W. Paine. Afterward, he established headquarters at New York, N. Y., but during the last ten years he had spent most of his winters in Fort Myers, Fla. It was at this place that death overtook him on December 1, 1933.

Mr. Dunham was a type of New Englander expressive of the best elements which have gone into the making of American civilization. He was independent in thought, original in conception, and fearless in the expression of his opinions as is amply shown by the part he took in the discussions of engineering subjects. He was a frequent contributor to the publications of the Society.

His rugged individualism was tempered by an unusual degree of modesty. His retiring nature was the cause of the uncertainty in which much of his life is clouded from the knowledge of his friends; but wherever portions of it became uncovered there came to the surface deeds of unselfish devotion and generosity. His interest in those whom he liked never ceased and extended to their children upon whom he lavished a true love. His life was one of service to others and was rounded out by his will, by which he deeded a small library which he built and equipped in his home town of West Chesterfield, to the community.

It is to be seriously regretted that a man of Mr. Dunham's fine analytical mind, high ethics, and love of home life, should have remained unmarried. He will continue, however, to live in the memory of those who have been privileged to know him.

He was a member of the Boston Society of Civil Engineers and of the New England Water Works Association.

Mr. Dunham was elected a Member of the American Society of Civil Engineers on October 1, 1890.

LEICESTER DURHAM, M. Am. Soc. C. E.[1]

Died November 21, 1933

Leicester Durham, the second of the four sons of Caleb Wheeler and Clarissa Safford (Welles) Durham, was born in Chicago, Ill., on May 4, 1877.

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[1] Memoir prepared by Henry Welles Durham, M. Am. Soc. C. E., with the assistance of William J. Shea and Arthur W. Tidd, Members, Am. Soc. C. E.

His ancestry on his father's side was descended from Andrew Durham, of Forfarshire, Scotland, whose son moved to Ireland in 1690, and whose great-grandson, John, came to New York, N. Y., in 1735. Leicester Durham's father was the youngest son of Alpha Durham, of Reading, Pa., whose wife, Elizabeth Riggs, was the daughter of Silas Riggs, of New Jersey, and of Harriet Rose, whose father, David Rose, of Southampton, Long Island, served as a private in the Revolutionary War and as an officer in the War of 1812. Mr. Durham's mother was the eldest daughter of Henry Woolsey Welles, of Ann Arbor, Mich., whose grandfather, Dr. Noah Welles, of Stamford, Conn., was the first minister of the Congregational Church there and a Chaplain in the Revolutionary War. His maternal grandmother was Susan Safford Welles, daughter of John Safford, a physician and merchant of Watertown, N. Y., who, as a young man, served as an Aide to General Walter Martin in the War of 1812. His father was a Civil Engineer who studied at the University of Michigan, worked on railroad location in Illinois, Arizona, and New Mexico, and invented and developed the Durham System of house drainage.

Leicester Durham was educated in private schools at Mount Vernon, N. Y., and in the Berkeley School, New York City, from which he was graduated in 1895. He then entered the School of Engineering at Columbia College (now Columbia University), New York City, from which he was graduated with the degree of Civil Engineer in 1899. While at college he became a member of Beta Theta Fraternity.

His first employment was with the Tubular Despatch Company, builders of the first pneumatic mail tubes in New York City. Afterward, he was with the Delaware Lackawanna and Western Railroad Company, on location surveys for the construction of new bridges over the Hackensack and Passaic Rivers. In 1900, after passing a Civil Service examination, he was appointed an Assistant Engineer with the Rapid Transit Commission of New York on the commencement of the first New York Subway, and served in the Third Division under the late Beverly R. Value and Cornelius Van Vorst Powers, Members, Am. Soc. C. E., Division Engineers, on the deep tunnel construction under Upper Broadway. On this work, one of Mr. Durham's outstanding characteristics was soon recognized by his superiors—his accuracy and attention to detail. This recognition was shown by increasing responsibilities given him in the precise survey work necessary in keeping correct alignment in headings run from offset shafts.

In September, 1905, he secured a transfer to the Engineering Department of the newly created Board of Water Supply, and was assigned to the Northern Aqueduct Department under Robert Ridgway, Past-President, Am. Soc. C. E., then District Engineer, comprising the Catskill Aqueduct line from Esopus Creek in Ulster County, south to and across the Hudson River to a point at Hunters Brook, about five miles southeast of Peekskill, N. Y. He was appointed Chief Assistant in the section under the late A. A. Sproul, M. Am. Soc. C. E., between Cold Spring, N. Y., and Hunters Brook. The first year's work was principally the making of location surveys and maps. Most of the personnel consisted of city boys with no field experience, and Mr. Durham's success in helping to weld them into an efficient working organization was noted by his superiors. Maps were rapidly produced, and the Chief Engineer, the late J. Waldo Smith, Hon. M., Am. Soc. C. E., referred to his work in complimentary terms.

On the commencement of construction in the following year, Mr. Durham was placed in charge of the section including the Valleys of Peekskill Creek and Sprout Brook, the Cat Hill Tunnel, and cut-and-cover work. It was a rough country, prices were low, and it was a discouraging task to carry on the work, but this did not have any effect on its quality, which was of a high order.

A partial re-organization of the Engineering Staff transferred him to the Walkill Division, west of the Hudson River, with headquarters at New Paltz, N. Y., under the late S. F. Thompson, Division Engineer. Mr. Durham had charge of the section of cut-and-cover work on the slope of the Shawangunk Mountains, under contract by the late James Pilkington. He remained on this work until its completion in 1914, when there was another re-organization, due to necessary elimination of supervision on the termination of construction of the Aqueduct, when he secured a transfer to the staff of the newly organized Bureau of Contract Supervision in the administration of Mayor John Purroy Mitchel. One of Mr. Durham's outstanding characteristics, in addition to his ability and integrity, was an inherent unwillingness to push himself forward or to secure recognition by the use of personal connections, not from any feeling of inferiority or shyness, but due to his dislike of self-seeking. This characteristic often prevented his obtaining the advancement which to many others seemed his due, but did not stand in the way of a general recognition of his value by those with whom he was long associated.

A little noted quality in Mr. Durham's nature was a strong underlying patriotism. As a college student in 1898, he was only prevented by his father's direct refusal of permission, from enlisting in the armed forces in the Spanish-American War. When the United States entered the World War, in 1917, he made immediate application for admission to the first Officers' Training Camp, starting in May, at Plattsburg, N. Y., and was commissioned a Captain in the Engineer Section of the Reserve Corps in August, 1917. Immediately thereafter, he was ordered to active duty as an Instructor at the second series of camps, beginning in September. No one could more justly have been considered exempt from any obligation to military service, except in an ultimate emergency, than he was at this time. He was married, with two small children, and a third was born during his tour of duty at Washington, D. C. His age also would have put him beyond any criticism, had he delayed offering his services. An unexpressed feeling that the members of a profession that was of peculiar value in war, owed something more than the ordinary, a sentiment of "noblesse oblige" that actuated many an engineer who would have denied its existence if questioned, undoubtedly was a motive, if an unadmitted one.

After a period of service in Washington and vicinity, Captain Durham was ordered overseas and served during the remainder of the war and until early in 1919, when he was returned and was discharged, as Personnel Officer of the 116th Engineers, a replacement organization for all engineer units, with station at Angers, France. He was assigned this duty because of his experience in the training and handling of new personnel and his knowledge of office organization and methods, and because an injury to his knee, resulting from an accident while on duty in Washington, had caused him to be reported unfit for active duty with combat troops, although in the course of his subsequent duties he made several trips to the front in command of units assigned for replacements.

After his discharge Mr. Durham returned to his post with the City of New York, which had been held for him in accordance with a regulation adopted in reference to all city employees serving with the armed forces of the country.

Early in 1919, following his reinstatement in the Bureau of Contract Supervision he was transferred to the office of the Chief Engineer of the Board of Estimate and Apportionment, and assigned to the field inspections made in connection with the various matters in regard to which the Chief Engineer was called upon to prepare reports. In 1922, he was detached from the outside work, and assigned to the investigations incident to many of the special projects that the Board of Estimate and Apportionment referred to the Chief Engineer, working under the personal direction of the Chief Engineer and the Consulting Engineer. Mr. Durham was engaged upon duties of that character continuously to the time of his death, the most exacting of which undoubtedly occurred during his last year. During that period he had general supervision of the preparation of projects for emergency work and relief throughout the entire city, in connection with the program instituted by the New York State Temporary Emergency Relief Administration as a part of the many efforts to relieve the acute nation-wide unemployment situation. His capacity for intense application was never shown to better advantage than during that period, when the pressure for relief called for tremendous efforts, and there is no question but that it seriously over-taxed his strength.

Mr. Durham had resided in Pleasantville, N. Y., after his return to civil life at the close of the World War, and during the intervening years he had been an active participant in the civic affairs of that community.

He was married on October 11, 1911, to Viola Ethel Hyde, of Peekskill, N. Y., and is survived by his widow and four children, Carol, Richard Hyde, John Dickinson, and Sarah Welles, and by two brothers, Henry Welles Durham, M. Am. Soc. C. E., and Knowlton Durham, an attorney of New York City. His body rests in Hillside Cemetery on a slope of the hill above Peekskill Creek, and within view of a section of the Catskill Aqueduct on the opposite hill, to the construction of which he contributed a substantial measure of personal skill and devotion. Those who knew him best would say that he was ever "all that is true, all that is brave."

Mr. Durham was elected a Member of the American Society of Civil Engineers on May 7, 1913.

FREDERICK JAMES EASTERBROOK, M. Am. Soc. C. E.[1]

Died March 30, 1933

Frederick James Easterbrook, the son of Nathan and Mary C. (Tester) Easterbrook, was born in New Haven, Conn., on December 11, 1870. New York was the native State of his parents, his father having been born at Herkimer and his mother at Little Falls. At the time of Mr. Easterbrook's birth, his father was connected with the United States Custom House in New Haven.

Mr. Easterbrook attended the public schools in New Haven, was graduated from the Hillhouse High School in 1889, and from the Sheffield Scientific School of Yale University, with the degree of Bachelor of Philosophy, in 1891.

After a year with the Edison General Electric Company, in Chicago, Ill., he entered the Engineering Department of the New York, New Haven and Hartford Railroad and remained with that Company until 1909. With some minor exceptions, his entire professional career of about forty years was spent in the broad field of railroad engineering. While a member of the Engineering Staff of the New York, New Haven and Hartford Railroad Company, the most important work on which Mr. Easterbrook was engaged was the double-tracking of the Naugatuck Division between Derby and Waterbury, Conn,; double-tracking on the Highland Division between Waterbury and Bristol, Conn.; and similar construction between Hawleyville and Danbury, Conn., a total of about forty miles, in which distance all grade crossings were eliminated and a number of new freight and passenger stations erected. The greater parts of these sections lie across main ridges and drainage lines of the territory traversed, thus necessitating deep cuts and high fills with numerous culverts and bridges. Mr. Easterbrook was in charge of these improvements.

For a period of six years, beginning in 1909, he served as Chief Engineer and Treasurer of the O'Brien Construction Company, of New York, N. Y. His principal activities were the building of the New York, Westchester and Boston Railroad between West Farms and Mount Vernon, N. Y., and the extension of the Grand Trunk Railroad from Blackstone, Mass., to Providence, R. I.

During the World War Mr. Easterbrook was associated with The American Brass Company, of Waterbury, Conn., in charge of the construction of factory buildings and other facilities for the production of war materials.

In 1921 and 1922, he was Construction Engineer with Westcott and Mapes, of New Haven. While associated with this firm he designed and installed the foundation for the Steel Point Power Station, at Bridgeport, Conn., for the United Illuminating Company, of New Haven.

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[1] Memoir prepared by Charles S. Farnham and E. W. Wiggin, Members, Am. Soc. C. E., assisted by John H. Myers, M. Am. Soc. C. E.

In 1923, Mr. Easterbrook was appointed Assistant Engineer of the Board of Estimate and Apportionment of New York City to investigate the structural condition of the elevated railroads. In this investigation he was closely associated with the late George Fillmore Swain, Past-President and Hon. M. Am. Soc. C. E., under whose direction the work was done. The following year Mr. Easterbrook took charge of the preparation of plans and estimates in connection with the West Side Improvements of the New York Central Railroad Company.

From 1925 to the time of his death, Mr. Easterbrook served as Assistant Engineer of the Board of Transportation, New York City, and assisted in the building of a section of the Eighth Avenue Subway along Central Park West from 89th Street to 100th Street—a four-track double-deck structure in an avenue flanked by heavy buildings and congested by a surface railroad and many sub-surface structures. Later, he served in a similar capacity on a section of three-track subway on the Grand Boulevard and Concourse, between 183d Street and Kingsbridge Road, in the Borough of The Bronx, New York City.

Mr. Easterbrook was a member of the Quinnipiack Club, of New Haven, and of the Yale Club, of New York City. He was also a prominent member of the Connecticut Society of Civil Engineers, having served as President of that Society in 1909.

He was married, in 1910, to Lucelia O'Brien, of New York City, who survives him.

Mr. Easterbrook was elected a Member of the American Society of Civil Engineers on May 3, 1905.

ARTHUR CHESTER EATON, M. Am. Soc. C. E.[1]

Died March 6, 1934

Arthur Chester Eaton was born at Lunenburg, Mass., on August 3, 1883. He was the son of Herbert Augustus Eaton and Anna Jeannette (Allen) Eaton, and a descendant, on his mother's side, of the Bradford and Washburn families famous in American history. His great-grandfather, Isaac Allen, served in the War of 1812. His grandfather, Isaac Allen, served in the Civil War, in the 53d Massachusetts Regiment, at Vicksburg and Port Hudson, Miss., was taken ill at Port Hudson and, dying on the way home, was buried at Memphis, Tenn.

Mr. Eaton attended the public schools at Lunenburg, and was graduated from Cushing Academy, at Ashburnham, Mass., in 1902, and from the University of Vermont, at Burlington, Vt., in 1907, with the degree of Bachelor of Science in Civil Engineering, *cum laude*. He was a member of Boulder, the Senior Honorary Society of the University, as well as Alpha Tau Omega

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[1] Memoir prepared by H. K. Barrows, M. Am. Soc. C. E.

Fraternity and Phi Beta Kappa. He was the representative of the Class of 1907 on the Alumni Council at the time of his death. In 1932, his Alma Mater conferred upon him the honorary degree of Civil Engineer, in recognition of his professional attainments.

Mr. Eaton's early professional work was in connection with the Catskill Aqueduct, of the New York, N. Y., Board of Water Supply, and in charge of New York State highway construction. In 1907 and 1908, he was, for a year, Instructor in Civil Engineering at the University of Vermont. From 1908 to 1912, he was engaged on water-power and storage investigations for the New York State Water Supply Commission and on the construction of the State Barge Canal, followed by water-supply and power work in New England.

In 1912, Mr. Eaton became associated with the Power Construction Company, the construction organization of the New England Power Company, which was beginning water-power developments upon the Deerfield River in Southern Vermont and Western Massachusetts. He became Assistant Hydraulic Engineer under the late A. A. Conger, M. Am. Soc. C. E., and was intimately connected with this important initial series of water-power developments of his Company, which has expanded into a system of 1 000 000 hp capacity, about one-half of which is water power.

In 1919, Mr. Eaton was made Hydraulic Engineer and in the next fifteen years had an active and important part in the long series of large water-power developments, which included the Searsburg, Harriman, and Sherman Plants, on Deerfield River, and the Bellows Falls, Fifteen-Mile Falls, and McIndoes Plants, on Connecticut River. These plants were examples of the highest skill and best types of engineering construction. For a paper of unusual excellence upon the Harriman Development,[2] Mr. Eaton, in 1924, was awarded the Desmond FitzGerald Medal by the Boston Society of Civil Engineers, of which he was a member.

Mr. Eaton's home was at Worcester, Mass., where he filled an important place in the civic and church life of the community, and was held in the highest regard by all who knew him. His sudden death on March 6, 1934, coming most unexpectedly, was a great shock to his family and his many friends.

He was married to Ethel Ida Obear, of Beverly, Mass., on June 30, 1909. Mrs. Eaton and two sons, Allen Ober and Chester Barstow, survive him.

The great esteem felt for Mr. Eaton by his associates, and the importance and value of his life work and recognition of his splendid character, were appropriately expressed in the resolutions passed by the Board of Directorate of the New England Power Company at the time of his death. which read, as follows:

> "Arthur C. Eaton, good friend and loyal worker through all the years of this Company, having passed away on the sixth day of March, 1934, this Board memorializes his passing by adopting this resolution.

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[2] "The New England Power Co. Davis Bridge Development," by Arthur Chester Eaton, *Journal*, Boston Soc. of Civ. Engrs., January, 1925, pp. 1-48.

"*Whereas,* Arthur Eaton, by reason of his exhaustive study of the New England rivers under all conditions of the changing seasons, spoke with authority on hydraulic matters pertaining to them, and by reason of his integrity and genuineness spoke with conviction.

"*And Whereas,* through the many years of his service he gave fully and freely to this Company of his knowledge and experience, and in many battles of litigation fought a difficult and winning fight.

"*And Whereas,* he was always a quiet doer of good deeds,—unselfishly living an upright and kindly life.

"*Be It Resolved,* that New England Power Company records its recognition and appreciation of the invaluable services and the fineness of the life of Arthur Eaton, and expresses deep sorrow and loss in his passing.

"*And Be It Resolved,* that a copy of this memorial be forwarded to the bereaved family."

Mr. Eaton was elected a Member of the American Society of Civil Engineers on July 6, 1920.

HARRY FRANKLIN FLYNN, M. Am. Soc. C. E.[1]

Died December 14, 1933

Harry Franklin Flynn, the son of John J. and Cordelia (Harrison) Flynn, was born at Woodbury, N. J., on June 14, 1869. During his early youth, the family moved to Ohio. His technical education was received at the Ohio State University, Columbus, Ohio, from which he was graduated in 1892, with the degree of Civil Engineer.

After a short period of employment as Draftsman for a bridge company, Mr. Flynn was appointed, in September, 1892, Computer with the United States Coast and Geodetic Survey in the main office at Washington, D. C. After four and one-half years of office work, during two and one-half years of which he was in charge of computations of all latitude observations, he was assigned to field work, in March, 1897, as Aide and Assistant. From that date until 1904, he was engaged on triangulation, base measurement, topography, hydrography, and astronomical work in the United States, Alaska, Hawaii, and the Philippine Islands, and was in charge during the greater part of the time.

In January, 1904, Mr. Flynn left the service of the U. S. Coast and Geodetic Survey to take a position with the Philippine Government in charge of survey work for the Bureau of Lands, Manila, Philippine Islands. While in this position he organized the work for some of the largest surveys, principally of "friar lands" acquired by the Philippine Government from Roman Catholic orders. From February, 1908, to July, 1909, he was Assistant Engineer, with the same Bureau, on road construction and maintenance, construction of concrete buildings, culverts, etc. He was District Engineer of Ilayan Province (about five months), Rizal Province, and Batanzas Province.

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[1] Memoir prepared by George F. Perry, U. S. Engr. Office, Philadelphia, Pa.

Mr. Flynn returned to the United States in 1909 and entered the service of the United States Engineer Department at Large, his first work being with the Seattle, Wash., Engineer District. From 1909 to 1916, he was engaged on surveys of Columbia River, in Washington, Gray's Harbor Bar, work in connection with the improvement of the Duwamish River, in Washington, and on surveys and plans for improvement of this and other rivers in the Seattle District. He was Resident Engineer on the Everett Harbor Improvement for one year and three and one-half years on concrete lock construction, for the Lake Washington Canal, under Arthur W. Sargent, M. Am. Soc. C. E. On this latter work he was in responsible charge of all layout, setting of gate pintles, trusses, sills, etc.

In November, 1916, Mr. Flynn took advantage of an opportunity to return East and accepted a position in the Philadelphia, Pa., District, U. S. Engineer Department, where he was in full charge of all survey work of the District in connection with contract and maintenance dredging. From 1923 to 1928, he was Principal Assistant Engineer in the Wilmington, Del., District, in which capacity he had immediate charge of the conversion of the old Chesapeake and Delaware Canal to a sea-level canal and other work in connection with river and harbor improvements.

In 1928, he was transferred back to the Philadelphia District where he assumed charge of research work, principally in connection with studies for the improvement of maintenance methods in the Delaware River. This work was carried on by a large corps of office and field assistants under his immediate supervision, and included exhaustive investigations of the discharge, tides, and currents in the river, silt movements, and analyses of saline content. Large-scale models of sections of the river were constructed under his supervision, for use in determining suitable locations for training works. Mr. Flynn was deeply interested in these investigations and by reason of his thorough mastery of mathematics and keen analytical mind he was peculiarly qualified to conduct them.

In addition to his work on the Delaware River project, he made studies of various proposed projects for harbor improvement and for the prevention of beach erosion on the Atlantic Coast. He also made hydraulic studies relating to the proposed canal across New Jersey from New York Harbor to the Delaware River. He was active in this work until his last illness. He succumbed to pneumonia on December 14, 1933.

Mr. Flynn was a member of the Society of American Military Engineers and of the Franklin Institute.

He was married at Seattle, on March 20, 1914, to Olivia Helen Case, who survives him. He is also survived by a brother, Oscar Roger Flynn, of East Orange, N. J.

Mr. Flynn was elected an Associate Member of the American Society of Civil Engineers on December 6, 1915, and Member on April 21, 1920.

ARTHUR DeWINT FOOTE, M. Am. Soc. C. E.[1]

DIED AUGUST 24, 1933

Arthur DeWint Foote was born at Guilford, Conn., on May 24, 1849. He was the son of George A. and Eliza (Spencer) Foote, and was born and spent his boyhood on the farm, "Nutplains," which had been in the family since 1750. He was one of ten children. After his public school education in Guilford, he entered Sheffield Scientific School, of Yale University, New Haven, Conn., in the Class of 1869, but a little more than a year after entering college, he was told by a physician that to continue his studies would destroy his eyesight. He, therefore, spent a year or more in Florida and other places. On discovering that the eye trouble was remedied by proper reading glasses, he resumed the study of engineering privately.

During 1874, Mr. Foote was an Assistant Engineer on the Sutro Tunnel which was being driven to drain and ventilate the famous Comstock Mines, of Virginia City, Nev. He used to tell of operating a transit, for a twenty-hour stretch, in a mine shaft in which the water was almost at the boiling point, and the mine pumps were operated by steam. He then went to California and was engaged on surveying in Golden Gate Park, in San Francisco, the Eldorado Canal, and on the location of the Southern Pacific Railroad across the Tehachapi Pass, under the late William Hood, M. Am. Soc. C. E.

Mr. Foote was Resident Engineer for the New Almaden Mine, in California, the largest quicksilver mine in America, from 1876 to 1877. He then tried unsuccessfully to secure capital for the building of a plant for the manufacture of Portland cement near Santa Cruz, Calif., where there is a good deposit of material, and where a plant has since been built. Failing this, he then secured the position of Surveyor at the Homestake Mine, at Deadwood, S. Dak., and, later (1879), became Superintendent of a mine at Leadville, Colo. He was with the United States Geological Survey, at Leadville, in 1880 and, in 1881, he made a trip to Mexico to report on some mines. Much of this traveling was by stage coach, and, in Mexico, it was all by diligence or on horseback.

In 1882, Mr. Foote went to Idaho as Chief Engineer of the Idaho Land and Irrigation Company, to build an irrigation system to cover 600 000 acres in the vicinity of Boise. Water was also to be supplied for placer mining along the Snake River. This was a great engineering project, involving storage reservoirs, dams, and two canals. The main canal was to have a capacity of 4 000 cu ft per sec, and to be 30 miles long. For a period of eight years, Mr. Foote persevered in efforts to raise the large amount of money necessary to finance this project. For headquarters and a home, with characteristic originality, he built a house near the site of the diversion dam, of the materials at hand, with walls 2 ft thick, of the rough basalt rock, using mud for mortar and for plastering the interior walls and partitions.

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[1] **Memoir prepared by Arthur B. Foote, M. Am. Soc. C. E.**

It was much more comfortable than a frame house, and saved the cost of a 10-mile haul for most of the material. Finally, the necessary money was subscribed, the smaller canal was built, and work had been carried on for nearly a year in the building of the main canal, when the Company became bankrupt due to a financial crisis. The late Andrew Jackson Wiley, M. Am. Soc. C. E., began his distinguished career as Assistant Engineer on this work. The project, which included the Arrowrock Dam, was afterward completed by the United States Reclamation Service, as the Boise Project.

In 1893 and 1894, Mr. Foote served as Chief Engineer of the Snake River Division of the U. S. Reclamation Service, and located a number of canals and reservoir sites which have since been constructed. In 1894, he opened an onyx quarry in Lower California where there was little water and no harbor. This involved transporting the onyx to the ocean and loading steamers at sea.

In 1895, Mr. Foote was engaged by his brother-in-law, Mr. James D. Hague, to build a power plant for a mining project in Grass Valley, Calif. Mr. Hague had organized the North Star Mines Company, and had acquired the North Star Mine and adjoining properties. None of these properties had ore in sight, and all but the North Star were closed down and full of water. Mr. Hague, therefore, showed most unusual confidence in his own judgment that these veins would prove profitable in depth, and the investors in the enterprise must also have had confidence in him.

As a preliminary, Mr. Foote was sent to investigate some recent mine-pumping plants driven by electricity in the Lake Superior copper mines. Electric power was comparatively new and not very well understood, and Mr. Foote decided against its use and adopted compressed air for the transmission of power. This involved the construction of a pipe line to bring in water under a head of 775 ft, the installation of a compressor with a 16-ft Pelton water-wheel mounted on the crank-shaft, all built for permanence, using masonry for buildings and for carrying the pipe across the creek. This type of construction was an innovation, except possibly in the case of a producing mine of proved value. Mr. Foote was then placed in charge of operations. Many difficulties were encountered in unwatering old mines, sinking a 1 600-ft vertical shaft to tap the vein far below the old workings, and it was not until 1902 that the enterprise began to pay a handsome profit. Mr. Foote remained in charge of the mines until 1913, when it became too difficult for him to go underground. He then resigned as General Manager and served as Consulting Engineer until a year before his death. During this period of 34 years the mines produced $30 000 000, and the North Star was deepened from 2 300 to 9 000 ft on the vein. For many years, either the North Star, or its neighbor, the Empire, has been the largest producer of gold in California. The Pelton water-wheel has been running night and day since 1896, but is now driving a new compressor.

Throughout his career Mr. Foote was a true pioneer. He was never afraid of trying something new. With the North Star Mines Company, he was the first to use small, one-man machine drills in the narrow stopes characteristic of the district, and developed a light, hard-hitting drill which was made in the mine shops. This proved the best drill for the purpose until the hammer drill came into use. The system he developed for extracting ore from the small veins having a pitch of only 30°, by means of the "go devil," which he invented, has been adopted wherever veins of that kind are worked. Under his direction, with Mr. E. L. Oliver as Metallurgist, a cyanide process was developed, after several years of experimental work, that solved the problem of extracting the gold from the mill-tailings economically, and the Oliver filter was the result. This filter is now in use in many parts of the world. The first delay-action electric exploders, made at the suggestion of Gerald Sherman, the Assistant Superintendent, were tried out at the North Star. Mr. Foote was the first in the district, and among the first in the country, to mount the mortars of stamp mills on masonry foundations. In 1904, he built a 40-stamp mill entirely of masonry and steel, except the frame of the building. This was a radical departure from current practice and saved much expensive repair work.

In 1908, Mr. Foote became interested in the development of a hydroelectric power project to furnish electric power for the mines. He directed the making of the necessary surveys and built a transmission line to connect the mining district of Allegheny with the Pacific Gas and Electric Company's system. Before construction of the power project had begun, the Pacific Gas and Electric Company reduced the rate for power to such a point that there was no economy in building the plant, and, therefore, the project was abandoned.

In 1909, a paper by Mr. Foote entitled "The Redemption of the Great Valley of California," was published by the Society.[2] Hydraulic mining had been stopped by law because the river channels were being filled with débris, causing them to overflow and flood the valleys. This paper described a project to allow the resumption of hydraulic mining and to dispose of the débris by filling in the swamps and lowlands in the Valley of the Sacramento and San Joaquin Rivers, thus renewing the fertility of the soil and gradually raising the land above high water. If it could have been carried out, it would have meant hundreds of millions of dollars added to the wealth of California; but it required the co-operation and consent of all the land owners in the Valley, which was impossible to obtain.

Mr. Foote could never accumulate money for himself. He was always supplying money to some prospector to develop a mine. In 1913, he was President and Manager of the Tightener Mine, situated in Allegheny, 40 miles from Grass Valley. The mine was only reached by a frightful road, part of which was 1 000 ft above its destination, and closed by snow in winter. Mr. Foote conceived the idea of building a new and shorter road at a lower elevation. Being unable on account of infirmity to go over the route himself, he located it on the small-scale Government topographic map and entrusted the staking out of the road to a local surveyor, who also made the estimate of cost. This man did not use common sense, and routed the road along the face of a cliff 600 ft above the river, when an equally good location was available higher up, over comparatively easy ground. Mr. Foote had obtained the estimated amount of money, necessary to build the road, from the two Counties, the Forest Service, and some of the mining companies, but this was not nearly enough for blasting a 15-ft shelf and building up rock retaining walls along the face of the cliff. Having contracted to build the road, he finished the job at a cost of about $50 000 of his own money, which was all he had. It remains an enduring monument to him, and is of great material benefit to the district it serves, as well as a wonderful scenic attraction.

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[2] *Transactions*, Am. Soc. C. E., Vol. LXVI (March, 1910), p. 229.

Mr. Foote always received the utmost loyalty and esteem from his assistants and employees. Many young mining engineers received their first practical experience at the North Star Mine. What they thought of their "boss" is well expressed in the following quotation from one of many similar letters received since his death:

> "All of us who started our mining experience at the North Star, which I have always regarded as a mining institution rather than as a gold mine, hold him in affectionate remembrance. His fine kindly character and high ideals permeated the whole North Star organization."

In 1876, Mr. Foote was married to Mary Anna Hallock, of Milton, N. Y., an illustrator and author, who survives him. He took her first to New Almaden, Calif., where their son was born, and, from that time, she accompanied him all over the West, traveling on horseback in Mexico, living in a log cabin in Leadville, and seven miles from the nearest neighbor in Idaho. Three children were born to them, of whom two are living, Mrs. Rodman Swift, of Hingham, Mass., and Arthur B. Foote, M. Am. Soc. C. E., of Grass Valley, Calif. A daughter, Agnes, died in her eighteenth year.

Mr. Foote was an incessant reader and during his lifetime collected a valuable library on irrigation, engineering, mining, and historical subjects. He was a member of the Mining and Metallurgical Society of America, the National Geographic Society, the University Club of San Francisco, and the Engineers' Clubs of San Francisco and of New York, N. Y., and a number of others.

Mr. Foote was elected a Member of the American Society of Civil Engineers on May 7, 1884. He served as a Director of the Society from 1910 to 1912.

JOHN PEDEN GARDINER, M. Am. Soc. C. E.[1]

DIED JUNE 15, 1931

John Peden Gardiner was the son of William and Barbara (Peden) Gardiner. His father went to Australia as a young man from Ayr, Scotland, and became interested in gold mining. He returned to Scotland about 1862, to marry, and returned to Australia soon after with his bride, on a sailing vessel which required three months to make the voyage. The couple settled at Creswick, Victoria, a small, but rapidly growing, pioneer village, and William Gardiner was soon made its Mayor. It was here that John Peden Gardiner, the sixth of seven children, was born on November 7, 1872. The family soon after moved to Geelong, Victoria.

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[1] Memoir prepared by Robert Ridgway, Past-President, Am. Soc. C. E.

After a preliminary education in the public schools of Geelong and Geelong College, John Peden Gardiner entered the University of Melbourne at Melbourne, Victoria, from which he was graduated on March 16, 1895, with the degree of Bachelor of Civil Engineering. During several months of his vacations in 1893 and 1894, he had worked with the Municipal Surveys in Melbourne, on general engineering work, under the local Government of Victoria. Shortly after graduation, he left for the United States, and as soon as possible thereafter, took out naturalization papers and became a citizen.

Mr. Gardiner was employed from October, 1895, to February, 1896, in an architect's office in Los Angeles, Calif., and, then, until November, 1896, with Arthur S. Hobby, M. Am. Soc. C. E., at Ontario, Calif., on general engineering work. From November, 1896, to June, 1897, he was engaged in mining surveying around Randsburg, Calif. Following this, he worked until March, 1898, as an Assistant to Mr. F. C. Finkle, of San Bernardino, Calif., on electric light and power work, and from then until December, 1898, he was with the Southern California Power Company as Assistant Engineer on the construction and concrete lining of $2\frac{1}{4}$ miles of tunnel and $\frac{3}{4}$ mile of flumes and accessory work, the cost of which amounted to approximately $400 000.

From December, 1898, to June, 1899, Mr. Gardiner served as Assistant Engineer on the construction of the railroad between Kingman and Chloride, Ariz. He then entered the service of the Oregon Short Line Railroad Company and was employed in office and field work until September, 1901. Between September, 1901, and June, 1903, he was in charge of the office at Springer, N. Mex., and was engaged in building the Dawson Railway, 132 miles long, and putting in concrete foundations for its steel bridges, including the one over the Canadian River; he was also in charge of the office at Alamogordo, N. Mex., for the El Paso Northeastern System.

From June, 1903, to March, 1905, Mr. Gardiner was employed by the Denver, Northwestern and Pacific Railroad Company, commonly known as the "Moffat Line," on field and office work. For a short time, from March to September, 1905, he was with the United States Reclamation Service, in charge of the design and laying out of the canals on Divisions 2 and 3, of the Huntley Project, in Montana.

In September, 1905, he came East and until March, 1906 was engaged as Assistant Engineer in the Designing Department of the New York Central Railroad Company, in New York City. Later, he was put in charge of a party engaged on revisions and in making surveys and estimates for several improvements and cut-offs for that System.

Mr. Gardiner then went to Durango, Dgo., Mexico, for the Guadalupe Mine, and from March, 1906, to June, 1907, was in charge of the layout and construction of a power plant, transmission line, and water supply for a mill and cyanide plant. The estimated cost of this work was about $185 000. He was in the City of Mexico when the Annual Convention of the Society was held there in July, 1907, and was quite active in entertaining the members from the States who attended it. It was there that the writer first met Mr. Gardiner. The acquaintance ripened into a friendship that continued until Mr. Gardiner's death.

After leaving Mexico, and until June, 1909, he was engaged in private practice as an Engineer in Los Angeles, Calif. For about a year following he made reports and surveys of a general engineering nature, and was associated with Manifold and Poole, Consulting Engineers, on the construction of dams and appurtenant work on Bishop Creek, California, for the Nevada-California Power Company. He also laid out and was in charge of the construction of a power plant for the Hydro-Electric Company of Bodic, Calif., estimated to cost $300 000. About 1910, he retired from the active practice of engineering, in order to devote his time to his properties and to personal affairs.

Mr. Gardiner was fond of ranching. A few years before his death, he purchased several hundred acres of land in the vicinity of Riverside, Calif., established an irrigation system for it, and planted it with a large number and variety of fruit trees. He took great interest and pride in developing them, and they were coming into good bearing when his untimely death occurred.

His personality and character were such as attracted good men to him, and the friendships he made were lasting. His interests were broad and varied, and it is difficult to emphasize any one of them as a particular "hobby." He was of an artistic temperament, and his charming home in Santa Monica, Calif., showed it. He excelled in amateur photography, and was an art collector to the extent that his opportunities permitted. Greatly interested in travel, he liked to study the habits and customs of all peoples. Grand opera and other good music appealed to him, and this led him to become a member and an enthusiastic supporter of the Gamut Club, a musical organization of Los Angeles. Mr. Gardiner was a member and one of the Judges of the Otis Art Institute of Los Angeles. He was also a member of the Jonathan and Athletic Clubs, of Los Angeles, and of the Brentwood Country Club and the Deauville Beach Club, of Santa Monica. He was a Thirty-second Degree Mason, and one of the founders of the Masonic Temple in Santa Monica.

Mr. Gardiner also took an active interest in the civic affairs of Santa Monica, and, for a time, was President of its Chamber of Commerce.

A spiritual touch in his nature made him very sympathetic to any one in distress. Charitable in thought and deed, he was generous to a fault, respected and loved in every community in which he lived.

On December 26, 1906, he was married to Virginia Melissa Bowman, of Los Angeles, who survives him. They had no children, and the bond between them was unusually close.

Mr. Gardiner was elected an Associate Member of the American Society of Civil Engineers on March 7, 1906, and a Member on September 6, 1910.

CARL GAYLER, M. Am. Soc. C. E.[1]

Died September 1, 1933

Carl Gayler was born at Stuttgart, Germany on October 22, 1847, the son of Julius Noah and Katherine Gayler. He received his technical education at the Polytechnic School of Stuttgart, from which he was graduated as a Civil Engineer in the summer of 1868.

After his graduation, Mr. Gayler was in the employ of the Government on a railway survey in Würtemberg, Germany. In April, 1869, he came to the United States and settled in St. Louis, Mo. At that time the disturbing effects of the Civil War had been fairly liquidated, and the West and Southwest for which St. Louis was the distributing point, were growing in population, in wealth, and in energy.

It was at this time that the late James Buchanan Eads, M. Am. Soc. C. E., had conceived the idea of spanning the Mississippi River at St. Louis. Conservative engineers were doubtful of the project, but the West was clamoring for men and materials and the East was ready to supply them. The unbridged Mississippi was a hindrance to progress and, therefore, must yield. Self-taught, Captain Eads required trained technical skill, and when he was ready he called to his aid several trained engineers to assist in building his bridge. Among these men was Carl Gayler, then 22 years of age.

From May, 1869, to April, 1871, Mr. Gayler was engaged in the office of the St. Louis Bridge Company by Captain Eads and served as Technical Adviser on the construction of the Mississippi River Bridge. "Technical Adviser" on that job meant a great deal more than knowing the processes and means of bridge work. Captain Eads' ideas and the nature of the problem called for what was then considered an almost impossible accomplishment in the way of new construction. To carry out the plans meant that new means must be discovered to overcome never-before-encountered difficulties, and new processes for the production of elements and members were necessary. The engineers of that day did not have at their disposal the tremendously efficient steel plants of to-day. Many of the processes that are now accepted as every-day facts were then unknown, and the fabrication of many necessary parts for the bridge hopelessly beyond the capacity of the then existing mills. The modern engineer who will picture the difficulties he would encounter if he were compelled, say, to discover a new foundry technique before he could proceed with construction, will get a rough idea of the position of the "Technical Adviser" to Captain Eads. Yet the bridge was built and stands to-day as a monument to the genius and skill of the engineers who carried it through.

From April, 1871, to January, 1872, Mr. Gayler was engaged on designs of passenger and freight stations for the Pittsburgh, Fort Wayne and Chicago Railway Company.

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[1] Memoir prepared by Walter B. Simon, Esq., University City, Mo.

He returned to St. Louis in January, 1872, and was employed by the engineering firm of Flad and Pfeifer, until December, 1874. From August, 1875, to December, 1876, Mr. Gayler served as Assistant Engineer to the late Henry Flad, M. Am. Soc. C. E., on the laying out and engineering construction of Forest Park in St. Louis, mostly on the design and construction of park bridges.

After a short engagement as Assistant Engineer with the North Missouri Railroad Company, Mr. Gayler was appointed in September, 1877, as Surveyor, and, after a few months, as District Engineer, of the City of St. Louis. In this position he was in charge of the repair and reconstruction of old bridges and the design and building of new highway bridges and culverts for the City. After the completion of the Jefferson Avenue Viaduct from his own design and under his direct supervision, the office of Bridge Engineer of St. Louis was created and Mr. Gayler was appointed to that position which he held for twenty-six years.

Mr. Gayler's many contributions to the growth of St. Louis are marked in granite and steel throughout the city. The Grand Boulevard Bridge, often spoken of as one of the most beautiful in America, was his work, as well as numerous viaducts across Mill Creek Valley. In speaking of his own work, the Grand Boulevard Bridge, he was highly critical, and once said with characteristic modesty and candor that he himself did not like it for a long time, but as the years went on, had permitted himself the opinion that perhaps the middle span was "right nice," although he really felt that common sense might question the building of a 500-ft span over "six or eight railroad tracks." "However," he added, "the bridge has carried the traffic for nearly forty years and without doubt can carry it for another forty, if properly taken care of, even in our smoky atmosphere; and that is more than can be said of most metallic bridges."

In 1914, Mr. Gayler engaged in private practice as a Consulting Engineer and was vigorously active until his retirement in 1928 at the age of 81.

At the time of his retirement, Mr. Gayler was honored by a banquet given for him by the St. Louis Section of the Society and the Circle Club, at which he made one of the first public addresses of his career. As the last living engineer on the famous Eads Bridge he reviewed in this address some of the personalities and problems of that great work. He spoke in terms of great respect and admiration of Captain Eads, who, he said, had had only a high school education and had never built a bridge before. Many people thought of Captain Eads, he continued, only as a promoter and financier, but he was also a great engineer and artist. Captain Eads himself, Mr. Gayler stated, designed the famous main arches of his masterpiece down to the smallest details of tubes and couplings, pin connections, skewbacks, and anchor-bolts. A bit of inside engineering history came to light when Mr. Gayler told of how the artist in Captain Eads caused him to make one mistake. The river piers bend. The east pier bent the day of the opening when ten test train loads were applied, so that to this day slight shakings may be felt to the east, west, north, and south, in the middle of one of the 500-ft spans. The tremors, although not serious, made it impossible to determine the stresses in the steel arches, and thus the key to the strength of the arches was lost. "This was caused," said Mr. Gayler, "because the artistic side of Eads was offended by the thickness of the piers as originally planned." Mr. Gayler said that he himself had seen Captain Eads come to the drawings, and slice off 2 or 3 ft of thickness with his pencil, in an effort to obtain more graceful design.

On November 7, 1871, Mr. Gayler was married to Minna von Rotteck, of St. Louis, who died in 1923. He is survived by two daughters, Caroline Gayler and Mrs. Walter B. Simon, and two sons, Julius F. Gayler, a Registered Architect, practicing in New York, N. Y., and Captain Ernest R. Gayler, C. E. C., U. S. N., M. Am. Soc. C. E. Another son, Dr. Wenzel C. Gayler (Past-President of the Missouri Medical Society), and a daughter, Mrs. Edward M. Harford, died before their father.

Always severely critical of his own work, but full of generous praise for the work of others when he felt it deserving, and known as a man of sincere, kindly feelings, Mr. Gayler enjoyed the respect, confidence, and affection of his colleagues and associates. He had added to the publications of the Society by numerous discussions on different types of bridges and viaducts.

Mr. Gayler was elected a Member of the American Society of Civil Engineers on September 3, 1884.

SAMUEL MARTIN GREEN, M. Am. Soc. C. E.[1]

DIED MARCH 22, 1934

Samuel Martin Green was born at Benton Harbor, Mich., on April 13, 1864. His father, Martin Green, was at that time a well-known Civil Engineer who was identified with some of the greatest engineering projects undertaken in the United States from 1850 to 1905. His uncle, Andrew H. Green, was for many years prominent in the civic life of New York, N. Y. In that connection, he was largely responsible for the preservation of its present Central Park and was frequently referred to as the "Father of Greater New York."

Samuel Martin Green was graduated from Worcester Polytechnic Institute, Worcester, Mass., in the Class of 1885. From 1885 to 1899, he was a Draftsman with the F. E. Reed Company, of Worcester, the Deane Steam Pump Company, of Holyoke, Mass., the John T. Noye Company, of Buffalo, N. Y., and the Merrick Thread Company, of Holyoke, of which he became Master Mechanic in charge of installation of equipment and erection of a cotton mill. In addition to this position with the Merrick Thread Company, he also served as Consulting Engineer for various industries in Holyoke. In 1899, he was appointed Mechanical Engineer for the American Thread Company, and constructed a large number of buildings, including mills at Willimantic, Conn., Westerly, R. I., and Holyoke and Fall River, Mass. (expenditure directed, about $2 000 000). He was also Consulting Engineer for the Farr Alpaca Company, of Holyoke, the Union Metallic Cartridge Company, of Bridgeport, Conn., and numerous fiber, paper, manufacturing, and other companies in Massachusetts and Connecticut.

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[1] Memoir prepared by Harry B. Hopson, Esq., Springfield, Mass.

In 1904, Mr. Green designed and supervised the construction of an envelope factory for the United States Envelope Company, at Waukegan, Ill. This mill building was approximately 250 ft long and 90 ft wide, and three stories high. In 1908, he designed and supervised the construction of another factory for the same Company, at Springfield, Mass. This mill building was approximately 300 ft long, 80 ft wide, and five stories high. In connection with the latter mill, he designed and supervised the installation of a steam boiler and power plant and electrified the entire plant.

In 1906, Mr. Green designed a steam boiler and turbine plant for the Union Metallic Cartridge Company, at Bridgeport, Conn. The boiler plant was of approximately 2 000 boiler-hp, and two condensing steam turbines were installed of 750 kw each. When this work was completed, the Union Metallic Cartridge Company requested him to make an examination of its primer loading plants after a disastrous explosion. He revised the method of loading primers entirely; built a new primer loading plant; and devised an entirely new method of handling and mixing fulminate, which, to that time, had been an exceedingly dangerous operation. Since the revision of the method by Mr. Green the Company has never had an explosion in its plant.

About that time, the Company had a disastrous explosion in its powder storage farm where many tons of powder had been stored. Mr. Green was given the problem of designing a powder storage farm that would be safe. He designed roads, bunkers, and small storage houses for powder, and supervised the installation of this equipment. The Union Metallic Cartridge Company was the owner of the Remington Arms Company, at Ilion, N. Y. This latter Company was not making a profit, and Mr. Green was given the commission of investigating the entire process of manufacturing arms. He spent two or three years at the Remington plant, designing new equipment and re-arranging machinery, until, finally, it was operating on a profitable basis.

In 1909, Mr. Green severed his connection with the American Thread Company and removed to Springfield, Mass., where he opened an office to devote his time to his private engineering business. Among his more important clients during this later period was the Farr Alpaca Company, of Holyoke, which spent under his direct supervision and charge from $12 000 000 to $15 000 000.

In 1911, he was given a commission by Mr. Max Henkels, who had come to the United States from Germany to engage in the lace-manufacturing business, at Bridgeport, Conn. Mr. Green designed and supervised the construction of a weave shed (188 by 62 ft), for Mr. Henkel. In 1912, he designed a two-story and basement brick mill building (213 by 65 ft) for this plant; and, in 1915, a three-story building of reinforced concrete (200 by 80 ft). The design of this latter building was rather extraordinary because the posts had to be located so that Nottingham lace looms could be installed in the building. The center of the posts was 30 ft lengthwise of the building and 25 ft crosswise. The floors of the building were of the mushroom type of construction.

In 1912 and 1913, Mr. Green designed and supervised the construction of a paper mill for the Strathmore Paper Company, at Woronoco, Mass., together with its power plant, filter plant, etc. For this same Company, in 1923, he designed a reinforced concrete bridge across the Westfield River, a span of approximately 400 ft, supported by three high arches.

In 1915, he designed and supervised the construction of the plant of the Warner-Klipstein Chemical Company, at South Charleston, W. Va. This was a caustic-chlorine plant consisting of many buildings, including a power plant of about 2 000-kw capacity, a plant for the production of carbon tetrachloride, carbon bi-sulphide, chlorine, and caustic soda.

In 1918, Mr. Green was awarded the contract by the United States Government to design and supervise the construction of the large chlorine plant at the Edgewood (Md.) Arsenal. This chlorine plant was the largest of this character that had ever been built to that time. Its capacity was 100 tons of chlorine gas and caustic soda per 24 hr. The expenditure made on this plant by the Government was approximately $9 000 000.

Mr. Green's work consisted in designing all the buildings which were of brick and wood construction. This plant consisted of a building for chlorine cells, approximately 600 ft long and 80 ft wide; a salt-treating plant about 200 ft square; a boiler plant, of approximately 2 000 boiler-hp; an evaporator plant for salt caustic evaporation, containing eight double-effect evaporators; and a caustic fusion plant. This latter building was approximately 250 ft long and 60 ft wide. He designed all the machinery and supervised its installation, and was under contract with the Government to supervise the installation of all the equipment and the starting of its operation. This plant was successfully constructed, and operation was started before the end of the World War, or in approximately six months. The plant was accepted by the Government and all bills were paid before the Armistice.

In 1926, Mr. Green designed a new boiler plant for the Fitchburg Paper Company, of Fitchburg, Mass. This work consisted in a complete revision of an old boiler-house and the installation of one 1 000-hp, 250-lb pressure boiler, with complete pulverized coal equipment, superheaters, and piping system. In connection with this work he also designed a high-pressure steam line, with all the supports, about 1 200 ft long, running from this boiler plant to one of the other mills over a bridge, about 100 ft long, also designed by Mr. Green. This work was entirely under his supervision; he let all the contracts and supervised the completion of the work.

In 1927, he designed a new boiler installation for Joseph Bancroft and Sons Company, at Wilmington, Del. This work consisted in the removal of two old boilers and the installation of one 1 000-hp, 250-lb pressure boiler, together with all its auxiliary equipment, stokers, etc., and a revision of the boiler-house for this installation. During 1927 and 1928, Mr. Green also engineered and supervised the construction of the Integrity Trust Building in Philadelphia, Pa.

He was associated with many engineering and scientific societies—the American Society of Mechanical Engineers, Engineering Institute of Canada, Engineering Society of Western Massachusetts, and the American Geographical Society.

He was always intensely interested in governmental matters, civic, State, and National, as instanced by his membership in the Springfield, Mass., Chamber of Commerce, Springfield Taxpayers' Association (of which he was President at the time of his death), the Constitutional Liberty League of Massachusetts, National Economy League, and Navy League of the United States.

While not active in the Masonic Fraternity, Mr. Green was a member of all the branches of the York Rite and of the Scottish Rite, being a Knight Templar, a Thirty-second Degree Mason, and also a member of the Shrine.

A list of Mr. Green's interests gives but a faint idea of his many activities and sterling worth, which are best known to those who were intimately associated with him.

He was married on June 18, 1890, to Ida McKown, of Holyoke, who survives him, with their daughters, Mrs. Kenneth B. Page, of Longmeadow, Mass., and Mrs. Lydia G. Meadows, of Springfield, Mass.

Mr. Green was elected a Member of the American Society of Civil Engineers on July 1, 1908.

EDWARD BUCKINGHAM GUTHRIE, M. Am. Soc. C. E.[1]

Died February 28, 1933

Edward Buckingham Guthrie, the son of Solomon Sturges and Jane (Sherwood) Guthrie, was born in Zanesville, Ohio, on July 25, 1849. When the son, Edward, was two years old, his father settled in Buffalo, N. Y., where he was active in transportation, civic, and religious affairs. He was a descendant of Abraham Pierson, the first President of Yale College, which fact no doubt influenced the choice of his son's Alma Mater, since Edward was graduated at New Haven in the Class of 1871, with the degree of Bachelor of Arts. Later, he completed the second-year course of study at Rensselaer Polytechnic Institute, Troy, N. Y., with the Class of 1874, leaving the latter institution to become a Leveler on the location of the Buffalo and Jamestown Railway (now a branch of the Erie Railroad).

From 1874 to 1880, Mr. Guthrie served as Assistant City Engineer of Buffalo, in charge of sewer work, leaving this appointment to spend two years in the employ of the Gilbert Starch Works, of Buffalo, for which Company he was engaged on the construction of a factory at Des Moines, Iowa. The next three years were occupied as Senior Partner of the firm of Guthrie and Clifton, in the manufacture of steel and iron wagon axles. In January, 1885, Mr. Guthrie was again appointed Assistant Engineer in charge of the construction and maintenance of sewers for the City of Buffalo, and, from 1886 to 1896, he served as Deputy City Engineer. In July, 1896, he was appointed Chief Engineer of the Bureau of Engineering, of Buffalo, retaining such position until November, 1897. During his connection with the City Engineer's Office, Mr. Guthrie directed the construction of 260 miles of pavement, 300 miles of sewers, the Elmwood Avenue (stone arch) Bridge, the Michigan Avenue (lift) Bridge, and six smaller bridges over the Scajaquada Creek. In November, 1897, he was appointed Chief Engineer of the Grade Crossing Commission of Buffalo, which Commission was charged, by an enactment of the Legislature of the State of New York, with the relief of Buffalo from the evils of the many highway-railroad grade crossings then existing.

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[1] Memoir prepared by a Committee of the Buffalo Section, consisting of George S. Minniss and John T. Mockler, Members, Am. Soc. C. E.

Mr. Guthrie's first connection with grade-crossing elimination work in Buffalo was in 1888 when, as Deputy City Engineer, he was one of a Board of Engineers appointed to formulate a general plan for such eliminations so that when later he was appointed Chief Engineer of the Grade Crossing Commission he was eminently qualified to carry out the details of the general plan which the Board of Engineers had evolved. At the time he was appointed as head of the engineering work of the Grade Crossing Commission only three street eliminations had been carried out, but during his tenure of office, grade crossings at seventy-eight streets, involving an expenditure of more than $15 000 000, were eliminated. The resulting structures are splendid examples of their kind and are in themselves lasting monuments to his industry and technical skill. In 1923, the Grade Crossing Commission was combined with the Terminal Station Commission of Buffalo, and Mr. Guthrie remained as Engineer of Grade Crossings until his retirement for age on December 31, 1927.

He met his death on February 28, 1933, when he was run down by an unknown automobile driver. He is survived by his widow, Elizabeth Seabury Guthrie, and one son, Edward H. Guthrie.

At different times, Mr. Guthrie had acted as Consultant on paving, water supply, and grade-crossing problems for various municipalities and business organizations and was a frequent contributor to engineering magazines and technical society periodicals. His painstaking accuracy as an Engineer and his remarkably retentive memory for details have been an inspiration to many of the younger members of the profession who came under his influence during his fifty-five years of engineering activities.

At a meeting of the Common Council, of the City of Buffalo, held May 7, 1933, the following memorial on the death of Mr. Guthrie, was adopted unanimously:

"The members of the Common Council of the City of Buffalo have learned with deep regret and profound sorrow of the death of Edward B. Guthrie, our esteemed fellow citizen, who was successfully engaged in the practice of engineering for many years.

"In 1874, Mr. Guthrie was appointed Assistant City Engineer, and after serving a period of six years, severed his municipal connection and became active in private enterprise. In 1885 he again entered the city service as City Engineer and held the position for 12 years. In 1897 he was appointed Chief Engineer of the Buffalo Grade Crossing Commission and continued in that capacity until 1923 and retired in 1927. Mr. Guthrie was prominently identified with grade crossing elimination and was respected as an authority in these projects.

"*Resolved,* That this Council record its high appreciation of the distinguished, faithful and efficient public service rendered throughout Mr. Guthrie's life, and the sense of the great loss this city has sustained by his death; and be it further

"*Resolved,* That the Common Council of the City of Buffalo extends its sincere sympathy and consolation to the family of the deceased; that a copy of these resolutions be engrossed and transmitted to his family, and that, as a further mark of our esteem, when this Council adjourns this day, it adjourn in respect to the memory of the late Edward B. Guthrie."

He was a member of Phi Beta Kappa, Delta Epsilon, and Wolf's Head, at Yale University; former Vice-President of the Engineering Society of Western New York and, also, of the American Society for Municipal Improvements; and to the date of his death he was an active member of the Saturn Club of Buffalo and the Westminster Presbyterian Church.

Mr. Guthrie was elected an Affiliate of the American Society of Civil Engineers on September 3, 1884, and a Member on October 5, 1887.

WILLIAM HENRY HALL, M. Am. Soc. C. E.[1]

DIED AUGUST 14, 1933

William Henry Hall, the son of Henry A. and Lucy J. (Matteson) Hall, was born in Riverpoint, near Providence, R. I., on January 25, 1881. Moving to New Britain, Conn., with his parents during boyhood, he attended the local schools and was graduated from Sheffield Scientific School, at Yale University, New Haven, Conn., in 1902, with the degree of Bachelor of Philosophy.

Mr. Hall entered the employ of the New York, New Haven and Hartford Railroad Company where he remained for the first seven years of his career as an engineer. After working as a Rodman, he was promoted to Transitman and given immediate charge of double-tracking five miles between Derby and Waterbury, Conn., which was then known as the Western Division. From 1905 to 1909, he served as Assistant Engineer, being associated with the relocation and tracking of the Naugatuck Division, making preliminary surveys, and establishing the line between Seymour and Waterbury, Conn., and in immediate charge of a section seven miles long through Naugatuck, Conn., including the building of the passenger station, freight-house and freight yard, and the elimination of all grade crossings. Later, he supervised the layout for the new freight yard and freight-house at Seymour, as well as the new freight yard, reinforced concrete roundhouse, machine shop, and turntable at Meriden Junction.

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[1] Memoir prepared by Edwin N. Lewis, Executive Secretary of Washington Cathedral, Washington, D. C.

From 1909 to 1914, Mr. Hall was in private practice with Arthur W. Bacon, Assoc. M. Am. Soc. C. E. The firm known as Hall and Bacon did general engineering work, including property development, State road construction, concrete retaining walls, quarry layouts, foundations, piers, and similar undertakings.

Following the election of the Hon. George A. Quigley as Mayor of New Britain, the Public Works Department was re-organized and, on March 5, 1914, Mr. Hall was elected City Engineer and Director of Public Works. He was also placed in charge of water-works extension. He made a commendable record, many improvements in the street and sewerage system being planned and completed under his direction. The ramifications of politics never were to his liking, however, and for some time before he resigned in 1920 it was known that he was seeking a larger field in the practice of his chosen profession.

Reviewing this period in Mr. Hall's career, his friend and associate, Ernest N. Humphrey, President of the Board of Public Works, in New Britain, says:

> "His record as City Engineer stands out and shows him as one of the best men to ever serve the city in that capacity. We found no mistakes in his work while he was engineer and none has turned up in the thirteen years since his retirement. When he served he was the only engineer for the city, acting for the sewer department, water department, subways, and streets.
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> "Among his outstanding works was the surveying of the Burlington Water-Shed, * * *."

Upon retiring from his work for the City of New Britain where his father was one of the leading business men and public-spirited citizens, Mr. Hall organized the W. H. Hall Construction Company, Incorporated, Manufacturers and Contractors of "Amiesite" for paving streets, roads, and driveways. Sensing the possibilities of this type of pavement, he built up a successful business from the start, which the Amiesite Company recognized by giving him the rights for all New England. His achievements did not come, however, without meeting difficulties with courage and faith, as is shown by the following anecdote told by one of his associates:

> "Shortly after the opening of the season our one and only plant at Plainville was completely wrecked by a railroad locomotive in the process of switching cars. This happened early in our busiest period crippling us completely during the three best months of the year—July, August, and September.
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> "This unfortunate accident was enough to discourage most any man, but Will, outwardly unperturbed, was on the scene next morning making plans to rebuild the plant. This incident was typical of Will throughout the following years when there were many and seemingly overwhelming obstacles in the way of his ultimate success."

As the business of his Company developed, Mr. Hall found need for opening a plant at Quincy, Mass. as there was only one trap-rock quarry in that section of the State. Later, he bought a controlling interest in this plant, the Old Colony Crushed Stone Company. Besides installing new machinery, Mr. Hall was able to increase the production of this plant to six times what it was when he first took hold of it. In addition to being head of the Company which bears his name, he was also President of the Massachusetts Amiesite Company which has a plant at West Quincy, Mass.

He gave all his talent and energy during the last thirteen years of his life to the W. H. Hall Construction Company. The various units of this Company include: An "Amiesite" plant in Plainville, Conn., a "Colprovia" plant and a sheet asphalt plant in the same town; two sand-pits also in Plainville; and a plant in Middlefield, Conn.

Golf players who use the Stanley Municipal Course in New Britain are indebted to Mr. Hall for his generosity when the construction of the first nine holes was proposed. He offered to execute the necessary work at cost and derived a great deal of satisfaction in carrying out the plans without profit to his organization. He and his associates built the golf course in a record time of sixty days and at a saving to the city of approximately $12 000. They also constructed the Stanley Quarter Park with its roads, artificial lighting, athletic field, one wooden bridge, and the Hawley Memorial Bridge.

Up to a few months before he passed away, Mr. Hall gave careful attention to all the details associated with his rapidly expanding business ventures, although he had suffered a severe heart impairment during the summer of 1931. His family and friends realize now that he was critically ill during the last six months of his life, yet he did not confide this knowledge to them. He must have felt appreciably improved in health during his last few days as evidenced by his talking with his only son about resuming their golf matches in which they had won one year the "Father and Son" Trophy at the Wampanoag Club.

On Sunday, August 13, Mr. Hall retired early at his home, in West Hartford, Conn., after having visited his sister in New Britain during the afternoon. He passed away peacefully while he slept. The cause of death was pronounced to have been coronary thrombosis. The funeral service was held from his residence on August 16, and interment was in Fairview Cemetery, in New Britain, beside his father.

Mr. Hall is survived by his mother, Mrs. H. A. Hall, of West Hartford; his widow, Mrs. Belle Parkhurst Hall, formerly of New Haven, Conn., where they were married on April 22, 1911; one son, William Henry, Jr., who is a student at Dartmouth College; two daughters, Katherine Parkhurst, who is a student at the Yale Art School, and Constance Hall; two brothers, Curtis A. Hall of West Hartford, Vice-President and Assistant Treasurer of the W. H. Hall Construction Company, and Alan V. Hall also of West Hartford; and two sisters, Mrs. Ernest W. Christ, of New Britain, and Mrs. Edwin N. Lewis, of Washington, D. C., formerly of New Britain.

Mr. Hall was a member of the Shuttle Meadow Club of New Britain; the Wampanoag Club of West Hartford; New Britain Lodge of the Benevolent and Protective Order of Elks, and the Connecticut Society of Civil Engineers.

When the news of his death reached Mayor Quigley, he ordered the flag on the City Hall of New Britain placed at half-mast. The Mayor later authorized this tribute to Mr. Hall for publication in this memoir:

"Associated with William H. Hall as I have been for over twenty years, I can say with deep feeling that he was a man among men, generous to a fault, an honest and efficient public official, eminent in the Engineering Profession, and a very able business man.

"'He was a man of sterling integrity, loved by a large circle of friends, and respected by all who had the pleasure of his acquaintanceship.

"We were very close friends and these few lines little express my deep grief at his passing or my regard for all the fine qualities he possessed."

The feeling which Mr. Hall inspired among his friends and their warm regard for him are expressed in the following tribute by Mr. Albert L. Worthen, of New Haven, who had been associated with him in many business transactions:

"My acquaintance with Will extended over twelve to fifteen years. We met first socially and later became very close friends.

"He had the unusual ability of looking into the future with amazing accuracy. When he gave up his position as City Engineer of New Britain to take over the manufacture of 'Amiesite' this business had, up to that time, proven a miserable failure. He had a vision of the possibilities of 'Amiesite' as a cold patch in highway maintenance instead of its use as a competitive material in new construction. His judgment and vision were justified because 'Amiesite' became recognized as the most satisfactory cold patch material and the demand for it exceeded Will's most optimistic expectations.

"In later years he saw possibilities for a small crushing plant in Massachusetts where lack of capital had been retarding the expansion necessary to take advantage of the potential market. He entered this field and arranged for the necessary capital to increase production, with the result that this business became highly profitable.

"I have never known a man who was as fair in his business dealings. The fact that he always met the other fellow more than half way and still was successful won for him the admiration of all those with whom he did business.

"Socially, he was a prince of a fellow, always anxious to help a friend, generous and thoughtful. Personally, I feel it was a privilege in my life to have known him and to have been included among his close friends."

Mr. Hall was elected a Member of the American Society of Civil Engineers on November 27, 1917.

FRED FORBES HENSHAW, M. Am. Soc. C. E.[1]

Died December 26, 1933

Fred Forbes Henshaw, the son of Josiah I. and Mary (Robinson) Henshaw, was born in West Brookfield, Mass., on May 21, 1884. He was graduated in 1904 in Civil Engineering, with the degree of Bachelor of Science, from the Massachusetts Agricultural College, at Amherst, Mass.

Mr. Henshaw served as Draftsman with the Brown Hoisting Machinery Company, of Cleveland, Ohio, for about six months, from March to September, 1905. In September, 1905, he entered the service of the United States Geological Survey with which he was employed for twenty-three years until 1928, serving, successively, as Hydraulic Aide, Junior Engineer, Hydraulic Engineer, and Senior Hydraulic Engineer. From 1906 to 1910, he was in charge of stream gauging in Alaska, working in Alaska during the summers and in Washington, D. C., during the winters. From 1910 to 1928 he was District Engineer in charge of surface water investigations in the States of Washington (1910–1912) and Oregon (1910–1928). In 1928, he was transferred from the Geological Survey to the Federal Power Commission as Senior Engineer, and remained in this position until his death.

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[1] Memoir prepared by N. C. Grover, M. Am. Soc. C. E.

Mr. Henshaw's name appears as the author or the joint author of more than fifty reports on stream gauging as related to Alaska, Oregon, and Washington. In addition, he was the author of several special reports published by the U. S. Geological Survey, of which, "Water Powers of the Cascade Range," Part II,[2] and "Deschutes River, Oregon, and Its Utilization,"[3] are the most important. He was also the author of many unpublished administrative and analytical reports of the U. S. Geological Survey and the Federal Power Commission.

He had an exceptionally active and inquiring mind. He was an easy writer and so produced many reports which were characteristically straightforward in presentation, clear in expression, and convincing in conclusion. His opinions were often sought by engineers and were always respected. He was much liked by his associates in the U. S. Geological Survey and the Federal Power Commission. A pleasing personality, an alert mind, absorbing interest in a wide variety of subjects, and his fairness in discussion, made him a genial and delightful companion.

Mr. Henshaw took a keen interest in the activities of the Society, having served for many years as Secretary of the Portland, Ore., Section and for one year in the same capacity for the District of Columbia Section, in Washington, D. C. He was a member of the Washington Society of Engineers.

He was married on November 24, 1908, to Carrie E. Lamond, of Cleveland, Ohio. In addition to his widow and his two sons, Lamond F. and Jesse R., and one daughter, Margaret Jane, Mr. Henshaw is survived by his parents and a brother, Charles Henshaw, who reside in Templeton, Mass.

Mr. Henshaw was elected a Member of the American Society of Civil Engineers on August 9, 1920.

BERTRAM HENRY MAJENDIE HEWETT, M. Am. Soc. C. E.[1]

Died November 14, 1933

Bertram Henry Majendie Hewett was born on December 23, 1874, at Shalford House, Shalford, Surrey, England. He was a son of the late Sir George

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[2] *Water Supply Paper 313*, U. S. Geological Survey.

[3] *Loc cit.*, *344*, U. S. Geological Survey.

[1] Memoir prepared by W. L. Lowe-Brown and J. Vipond Davies, Members, Am. Soc. C. E.

J. H. Hewett, Bart., and his second wife, Sophia Henrietta (Godwin-Austin) Hewett. He was educated at Sherborne School and, subsequently, took the course in Civil Engineering at the City and Guilds (Engineering) College, London, England, from which he obtained his diploma.

From February, 1896, to March, 1898, Mr. Hewett was employed as a Junior Assistant Engineer by the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., Mr. J. H. Greathead, and Mr. Basil Mott (afterward Sir Basil) on the construction of the Central London "Tube" Railway. On this work he gained his first experience of tunneling which was destined to be his life work. He acquitted himself so well on his first engagement that when it was completed he was appointed, at the early age of 24, as Resident Engineer on an extension of the City and South London (Tube) Railway from Stockwell to Clapham Common, the engineers for this work being Sir Benjamin Baker, Mr. Basil Mott, and Mr. David Hay. When that work was completed, Mr. Hewett was engaged as Second Resident Engineer on the extension of the same railway from Moorgate to Islington. On the completion of these extensions in March, 1902, he returned to the offices of Messrs. Baker, Mott, and Hay to assist in the preparation of plans for further tube railways in London.

Early in 1903 he joined, as Surveyor, the Bullock-Workman Geographical Expedition to the Karakoram Himalayas, Baltistan, Kashmir, British India. This expedition made an extensive exploration of glaciers and mountains in that region, including ascents to more than 23 000 ft. Their discoveries are described in a book entitled "Icebound Heights of the Mustagh," by F. B. and W. H. Workman, published in 1908. Mr. Hewett was responsible for the map of the region contained in the book and for the research into the movements of glaciers described therein. His work was severely handicapped by the lack of trained assistance and by the frost-bite from which he suffered. He carried on with his survey work long after he had ceased to be fit and was finally brought back to the base with both feet severely frost-bitten and suffering from acute dysentery. His feet were only saved by keeping them for several weeks in cold water, continually replenished from the glacier.

On his return from India, Mr. Hewett accepted an appointment with the Pennsylvania Railroad Company under the late Charles M. Jacobs, M. Am. Soc. C. E., Chief Engineer of the North River Division, as Resident Engineer on the construction of two tunnels under the Hudson River, at New York, N. Y. These twin tunnels, 23 ft in diameter, were the essential link required to give the Pennsylvania Railroad Company a direct entrance with a terminal station in New York City. Previous to this time no tunnels under the Hudson River, at New York, had been completed, and several earlier attempts to tunnel the river had been unsuccessful. The difficulties consisted in the mixed nature of the ground through which the tunnels had to be constructed. Starting in solid rock, the tunnels gradually passed to a mixed face of rock, sand, and clay, and, finally, for the whole width of the river, nearly a mile, the material consisted of soft mud. The work is described in a paper[2] entitled "The New York Tunnel Extension of the Pennsylvania Railroad: The North

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[2] *Transactions*, Am. Soc. C. E., Vol. LXVIII (December, 1910), p. 152.

River Tunnels," by Mr. Hewett and W. L. Lowe-Brown, M. Am. Soc. C. E. For this paper the authors were awarded the Thomas Fitch Rowland Prize by the Society in 1911.

After the completion of the Hudson River Tunnels of the Pennsylvania Railroad Company, in February, 1910, Mr. Hewett's first work, in his new employment as a member of the staff of Jacobs and Davies, Consulting Engineers of New York City, was to visit India to collect the information required for three reports. The first for Mr. Ratan Tata concerned a variety of alternative projects for the reclamation of the Bombay Back Bay, in India, and other low-lying ground in the vicinity. The second report, also for Mr. Tata, drew up in some detail, a project for a tunnel under the River Hooghly at Calcutta, India, for a direct connection between the East Indian Railway at Howrah and the Eastern Bengal Railway at Beliaghata. The third report related to a proposal to introduce grain elevators into India, with special reference to the handling of wheat at the Port of Karachi. These three reports occupied practically the whole of the year 1910.

On his return to New York, Mr. Hewett was engaged on the preparation of various underground passenger transportation projects in New York City and Brooklyn, and also in the study of the complicated geological conditions which were the determining feature in the construction of the Astoria Gas Tunnel, in New York City, which his firm was building as designing and constructing engineers.[3]

Early in 1912 he was sent by his principals to Mexico to take charge of the construction of the Laxaxalpam Aqueduct and Tunnels. The project consisted of a series of twenty-six tunnels forming an aqueduct, 18 miles long, in the mountains of Puebla, to provide an additional supply of water to the Laxaxalpam Dam. These works had been started in 1909, but unforeseen difficulties had been encountered and, after five unsuccessful attempts to carry them out, construction had been brought to a standstill. Realizing that the only chance of bringing the work to a successful termination was to seek the aid of experienced tunnel engineers, the Mexican Light and Power Company, under the late Fred Stark Pearson, M. Am. Soc. C. E., called upon the firm of Messrs Jacobs and Davies to take over the task. Mr. Hewett was sent, in charge of an experienced staff, in July 1912, to make the necessary designs to meet the conditions, and to carry out the construction by "force account." In spite of labor troubles and political difficulties, due to the disturbed state of the country at that time, he finished the work by May, 1913, when only a few days' margin of water supply remained in the reservoir.[4]

On the completion of the Laxaxalpam Aqueduct, Mr. Hewett returned to New York City and took an active part in all the work of his firm. At first, he devoted most of his time to the design and construction of the subways and structures of The New York Municipal Railway Corporation, under its contracts with the City of New York for the construction and operation of rapid transit lines. As Chief Assistant, he took general charge of the work on those sections which his firm carried out as Designing and Constructing Engineers. This work involved a large amount of reconstruction and new construction to inter-connect the incomplete Fourth Avenue Subway, the "Brooklyn Fifth Avenue Elevated Line," and the two-track extension of the South Brooklyn Railway to Coney Island. At the same time, it was necessary to interweave all these lines with the two proposed new elevated lines from 36th Street, Brooklyn, to Coney Island. All modifications of the existing tracks were carried out without interruption to the heavy suburban traffic passing over them.

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[3] "The Astoria Tunnel Under the East River for Gas Distribution in New York City," by J. Vipond Davies, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 594.

[4] "The Laxaxalpam Aqueduct Tunnels, Mexico," by James Forgie, M. Am. Soc. C. E., *Minutes of Proceedings*, Inst. C. E., Vol. CC (1914–15), Pt. II, p. 345.

Mr. Hewett's next work was in connection with the reconstruction of the intake and the discharge tunnels for condensing water, for both the Waterside Station of the New York Edison Company, and the Williamsburg Power Plant of The New York Municipal Railway Corporation, to provide for the requirements of their greatly increased output. These works, involving the use of both compressed air and open coffer-dam construction, were carried out under his supervision without interference with the operation of the power plants.

Generally speaking, during the whole of the time from the completion of the Pennsylvania Railroad Tunnels until January, 1926, when he left New York City to take charge of the construction of the Mersey Tunnel, Mr. Hewett was associated as Managing Engineer and Director, with the firm of Messrs. Jacobs and Davies. His work throughout this connection was recognized as being of the highest value to the undertakings concerned. It involved a great breadth of understanding of many diverse branches of engineering, particularly of modern rapid transit railroad development, equipment, and operation under the latest conditions of electric traction. His grasp of the development of the Dual Subway System, in which The New York Municipal Railway Corporation represented a half interest, was complete and involved engineering ability of a very high order.

Early in 1926, Mr. Hewett was appointed Engineer-in-Charge of the construction of the Mersey Tunnel, working directly under Sir Basil Mott who was Engineer to the Mersey Tunnel Joint Committee, a statutory body charged with the carrying out of the project.

The Mersey Tunnel is primarily intended to connect Liverpool and Birkenhead and the docks on each side of the river by a roadway for vehicular traffic, but it also forms a through route between Lancashire and Cheshire The largest subaqueous tunnel yet built (being 44 ft in internal diameter, with a roadway sufficient for four lines of traffic), it has been constructed entirely through rock highly charged with water, the volume pumped being about 4 500 gal per min while the tunneling was in progress. The tunnel is lined with cast iron to make it water-tight and to obviate the necessity of continuous pumping. The fact that the tunnel will be used exclusively by motor vehicles, made the provision of a complete ventilating system essential to cope with the poisonous exhaust gases. The size of the plant required, may be gauged from the fact that, including reserves, it is capable of delivering 5 000 000 cu ft of fresh air per min and of exhausting the same volume of vitiated air. The ventilating machinery, the electric lighting system, and the auxiliary plant, essential for the efficient maintenance of the structure, have required an immense amount of detailed study and attention. Mr. Hewett gave himself unsparingly to the task, and no praise is too great for the skill he displayed in carrying out his duties or for his unfailing tact in co-ordinating the services of the numerous specialists and in dealing with the various contractors employed.

It is deeply regretted by all connected with the project that, by his untimely death at West Kirby, Cheshire, on the eve of its completion, Mr. Hewett should have been prevented from seeing the opening in 1934, of the work to which he had devoted the last eight years of his life.

Throughout the whole of his career, whether in England or America, Mr. Hewett was admired and beloved by all with whom he came in contact, not only for his great ability as an engineer and for his charming personality, but also for his courtesy, tact, and fair-mindedness. Owing to his modest and retiring disposition, he was not as well known as he deserved to be, but to those who had the privilege of his friendship, his death leaves an irreparable void. The Engineering Profession has lost one whose experience and knowledge of tunnel engineering was unique, whether measured by the length of time devoted exclusively to that type of work, or to the diversity of the natural difficulties of the ground which he had been called upon to overcome.

Fortunately, the results of his experience will not be entirely lost to the profession with his death, for in addition to the marks of his ability deeply imprinted on all the works with which he has been connected, he has left a record of at least a part of his experience, first in the paper describing the Pennsylvania Tunnels previously referred to, then in a joint paper[5] with S. Johannesson, M. Am. Soc. C. E., on "Notes on Tunnel Lining for Soft Ground," and, finally, in the authoritative work written in collaboration with Mr. Johannesson, entitled "Shield and Compressed Air Tunneling," published in 1922, in which are crystallized many of the results of his wide experience.

Mr. Hewett was a Member of the Institution of Civil Engineers, London, England. He was also a member of the Royal Geographical Society.

He was married in 1911, in New York City, to Mrs. Theodora (Mooney) Weir, who survives him.

Mr. Hewett was elected an Associate Member of the American Society of Civil Engineers on April 5, 1905, and a Member on October 1, 1907.

WARREN ALBERT HOYT, M. Am. Soc. C. E.[1]

Died October 18, 1933

Warren Albert Hoyt was born in Bradford, N. H., on September 2, 1868, the second of five children of Benjamin Frank and Alzada (Eaton) Hoyt. Mr. Hoyt was of early Colonial ancestry on both his father's and his mother's side, dating back to 1640.

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[5] *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1822.

[1] Memoir prepared by Mrs. Warren Albert Hoyt and F. J. Hoyt, Esq., Altoona, Pa.

His father died when he was 13 years of age with the result that he and his older brother had to assume the responsibilities of operating their New England farm. Thereafter, he was practically self-supporting, and it was necessary for him to turn to all manner of employment to obtain the education which he so much desired. Mr. Hoyt first attended the New London, N. H., Academy from which he was graduated in 1894. His mother, two sisters, and a brother having died by this time, and the health of his younger brother being impaired, they went to Colorado. While there he took a Liberal Arts Course at Colorado College, at Colorado Springs. After the completion of three years of this course, he decided to take up the study of engineering. For this reason, he came East to the University of Wisconsin, at Madison, Wis., from which he was graduated in 1900 with the degree of Bachelor of Science in Civil Engineering. As post-graduate work, he took up the study of Architecture at the Art Institute of Chicago, Ill.

Mr. Hoyt began his engineering career in the summer vacations during his attendance at the New London Academy. At this time, he was a member of a surveying party under the City Engineer, of Providence, R. I. His next experience was while he was attending Colorado College, when he was engaged on construction survey work for the Midland Extension Railroad, of Cripple Creek, Colo.

From 1900 until 1902, Mr. Hoyt was Assistant City Engineer of Altoona, Pa. During this time he was in charge of sewer construction, city paving, water supply surveys, septic tank experiments, etc. From Altoona he went to Sault St. Marie, Ont., Canada, where he was employed, for a time, by the Algoma and Hudson Bay Railroad Company, as Draftsman and Designer of bridge abutments, retaining walls, trestles, etc. From here, Mr. Hoyt went to Kalamazoo, Mich., where he was engaged with G. S. Pierson, M. Am. Soc. C. E., Consulting Engineer, as Assistant Engineer in charge of designing and estimating on sewers, paving, and similar work.

During the period from 1904 to 1906, Mr. Hoyt was Chief Engineer for the Shillinger Brothers Company, Contractors, of Chicago, Ill. A part of this time he served as Superintendent of Building and Bridge Construction; he was also in charge of estimating and bidding on bridges and buildings ranging in value from $50 000 to $500 000. Mr. Hoyt was next employed by the Corrugated Bar Company, of St. Louis, Mo., as Assistant Engineer in charge of estimating, and as Assistant Superintendent in charge of concrete work on the viaduct roadway from Kansas City, Kans., to Kansas City, Mo.

From 1906 until 1909, he was with the Corn Products Company, of Chicago, as Chief Concrete Engineer, in charge of designing buildings, foundations, subways, bridges, docks, reservoir, and grain elevators at the Argo, Ill., Plant. He also had general supervision of construction work on the plant. This plant was one of the largest undertakings in reinforced concrete at that time, there being approximately forty buildings ranging in height from one story to eight stories. The structural part of the buildings alone cost approximately $6 000 000. This construction comprised that of the first unit only, other units having been added at a later date.

Mr. Hoyt was next engaged, from 1909 to 1913, as a Consulting Engineer in Chicago. During this time he designed and constructed many buildings in and around that city. It was during this period that Mr. Hoyt was appointed by the United States Civil Service Commission as Designing and Constructing Engineer for a three-story reinforced concrete building at Rock Island, Ill., Arsenal. This was used for heavy ordnance, the guns being tiered on the floor.

From 1913 until 1916, he was associated with M. J. Morehouse, Architect, of Chicago. During this period he was in charge of engineering, designing, and some supervision of construction for many buildings. Mr. Hoyt then left Chicago and, from 1916 until his death, he was located at Altoona, Pa. As a Consulting and Structural Engineer, he designed and constructed many buildings in the cities of Central Pennsylvania.

In 1900, during the construction of various concrete warehouses and stores in Chicago, cold-weather tests were instituted and studies made to determine methods of handling reinforced concrete in cold weather. The results were published in various technical magazines. Later, these tests were instituted at the University of Illinois, Urbana, Ill., as thesis studies, all summarized in a paper by A. B. McDaniel, M. Am. Soc. C. E., entitled "Influence of Temperature on the Strength of Concrete."[2] While in Chicago, Mr. Hoyt wrote various articles for technical papers, and reviewed many books on reinforced concrete for the Western Society of Engineers.

Mr. Hoyt gave a great deal of his time to the service of the City of Altoona. He became a member of the City Planning Commission in 1920, and served continuously until his death. Early in his service he was elected Chairman of the Commission and retained this position until three years before his death, when ill-health made it necessary for him to relax. Mr. Hoyt was also a member of the State Association of Planning Engineers. As a member of the City Planning Commission, he had a leading part in the efforts which led to the adoption of the Zoning Code in force in the City of Altoona.

He was an omniverous reader. His personal library contained the best of books on many subjects. He never ceased to be a student after leaving college and he possessed a wonderful range of knowledge on practically every conceivable subject. Public education was one of his greatest interests, and he was active in promoting every movement with that end in view.

Mr. Hoyt was a member of many National Societies, among which were the American Concrete Institute, the American Association for the Advancement of Science, the Western Society of Engineers, the Rotary Club, and the Torch Club. He was also a member of the Altoona Chamber of Commerce and a number of small civic clubs organized for the purpose of achieving a broader outlook on life.

He was married to Mary Piper Jamison, of Altoona, on September 25, 1907. He is survived by his widow and two sons, Fred J. and Stephen H. Hoyt. His brother, Chester F. Hoyt, is also living, at Sacramento, Calif.

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[2] *Proceedings*, Am. Concrete Inst., Vol. 12, p. 241.

Warren Albert Hoyt was a respected and honest citizen. He was idealistic, far-seeing, of broad culture and outlook. His exceptional engineering abilities, combined with his intense public spirit, made him a civic leader.

Mr. Hoyt was elected a Member of the American Society of Civil Engineers on June 30, 1910.

ADAM HUNTER, M. Am. Soc. C. E.[1]

Died November 1, 1933

Adam Hunter was born at Crossford, Fifeshire, Scotland, on August 23, 1869. He came of an engineering family, his father having occupied an important position with Messrs. Tancred and Arrol on the construction of the Forth Bridge.

Adam Hunter developed a bent for engineering and as a young man joined Tancred and Arrol's Staff resident at the Forth Bridge as an Apprentice Pupil from 1886 to 1889, under the late Mr. J. C. Tuit. He was subsequently transferred to the London Office of the Company and took part from 1889 to 1895 as Assistant Engineer in the construction of the Tower Bridge, Shoreham Viaduct on the London, Brighton and South Coast Railway, and other works, again under Mr. Tuit. At this time, 1890 to 1893, he was also a student at the City of London College. From 1895 to 1905, as Chief Assistant Engineer, of Sir William Arrol and Company, Limited, he was occupied in the construction and erection of many important structures, bridges, power stations, factories, large cranes, dock-gates, etc., in Great Britain and Ireland, Africa, China, Japan, Greece, etc.

In 1905, Mr. Hunter was in Egypt as Technical Adviser for the Company on the construction of bridges over the Nile at Cairo. In 1906, he was appointed Chief Engineer to Sir William Arrol and Company, at the Headquarters Office, at Glasgow, a position which he held until his illness in the early part of 1932. He was also a Director of the Company and from 1906 onward he was responsible for the construction and erection of all the works carried out by the firm. A few of the more important of these are: The Barrow and Suir Viaducts, in Ireland; the Queen Alexandra Bridge; Middlesbrough Bridge; Blackfriars Bridge widening; Southwark Bridge; Waterloo Temporary Bridge; Prah River Bridge, on the Gold Coast in Africa; Keadby Drawbridge; dock-gates and caissons for the Royal Naval Dockyards, at Rosyth and Methil; Wearmouth Bridge; Athens, Greece, Power Station; dock-gates and swing bridge for Nag Hammadi Barrage; Benue Viaduct, Nigeria; Ribble Viaduct; 250-ton electric hammer-head crane, and a large number of other important structures in various parts of the world.

During the World War Mr. Hunter was responsible for important Government works carried out by the Admiralty, the War Office, and the Air Ministry. At the close of the war, he was appointed a Consultant Engineer by the War Graves Commission and visited France on several occasions in connection with this work.

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[1] Memoir prepared by George Moncur, M. Am. Soc. C. E.

Mr. Hunter took a warm interest in the affairs of the Institution of Civil Engineers and, in conjunction with Mr. J. C. Buscartlet, he presented a paper before its members on April 12, 1910, entitled "Queen Alexandra Bridge over the Wear at Sunderland."[2] He was especially interested in the younger members of the Institution and for two successive years was Chairman of the Glasgow Students' Branch. He had also served as President of the Glasgow University Engineering Society.

He was a Vice-President of the Institution of Engineers and Shipbuilders of Scotland, a former Vice-President of the Institution of Structural Engineers, and of the Institution of Junior Engineers, in London, before which on December, 1906, he read a paper on "The Structural Design of Engineering Workshops."[3] He was also a member of the Royal Philosophical Society of Glasgow, the Engineers' Club of London, and the Liberal Club of Glasgow. In addition to the papers mentioned, he was the author of a book entitled "Arrol's Bridge and Structural Engineer Handbook."

Mr. Hunter had a vast store of engineering knowledge which was always fully at his command, and his services on Engineering Committees and as a Consultant were held in high esteem. He was a member of a number of the British Engineering Standards Committees, and it was while presiding at a meeting in London of the Committee on Bridge Loads and Stresses that he was first stricken by the illness to which he finally succumbed at his home in Rutherglen, Scotland.

Of a generous and genial disposition, Mr. Hunter had hosts of friends both at home and abroad and was very highly esteemed by all who came in touch with him, not only for his great professional ability, but for his sterling worth and amiable personality. He is survived by Mrs. Hunter and a married daughter.

Mr. Hunter was elected an Associate Member of the American Society of Civil Engineers on September 6, 1905, and a Member on May 2, 1911.

CHARLES EDWIN JENKINS, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 29, 1933

Charles Edwin Jenkins, the son of David and Annie (Birtwistle) Jenkins, was born in Nutley, N. J., on August 20, 1876. After his graduation from High School, his technical education was achieved by a partial course at a technical school, and by evening study.

Mr. Jenkins' early engineering experience was in railroad surveying. From 1902 to 1904, he was with the Long Island Railroad Company on reconstruction of terminals, piers, ferry-slips, and special track work. In 1904, he became Assistant City Engineer of Easton, Pa., carrying on, in addition, outside work for an electric railroad line which led to his engagement, in 1905, by the Easton and South Bethlehem Transit Company, which later became the Easton Transit Company. Mr. Jenkins was with this organization until 1914 as Engineer of Maintenance of Way, in charge of the design and reconstruction of bridges, tracks, etc. For the three following years he was Chief Engineer of the Lehigh Valley Transit Company, of Allentown, Pa., which operated 250 miles of city and interurban railway.

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[2] *Minutes of Proceedings*, Inst. C. E., Vol. 182, 1909–10, Pt. IV.

[3] *Record of Transactions*, Inst. of Junior Engrs., Vol. 16, 1906.

[1] Memoir prepared by Prof. E. F. Church, Jr., The Polytechnic Inst., Brooklyn, N. Y.

Leaving this position to enter World War activities, he went with the American International Shipbuilding Company, in partial charge of the construction of ten concrete shipways, and, later, in charge of fifty shipways as Ship Engineer. Ill-health forced him to resign from this work and, later, he became Engineer of Tests, Production, and Inspection for the Air Nitrate Corporation. From 1919 to 1923, Mr. Jenkins was Manager of Engineering Development Work for the Railways Equipment Company, and the Odorless Incinerator Company, of Philadelphia, Pa., and for the four following years, for the New Mix Products Company.

In 1929, Mr. Jenkins formed a connection with Jensen, Bowen, and Farrell, of Ann Arbor, Mich., engaged in appraisal work of public utilities in various cities and States, in which work he was engaged at the time of his death.

He was married in 1899 to Rose B. Hankinson, deceased. There are three living children; Mrs. Margaret Louise Bruere, Edwin David Jenkins, and George Elbert Jenkins. He is also survived by his widow, Mrs. Helen B. Jenkins, to whom he was married in 1913.

Mr. Jenkins was elected a Member of the American Society of Civil Engineers on May 7, 1913.

JAMES MORELAND JOHNSON, M. Am. Soc. C. E.[1]

DIED FEBRUARY 11, 1934

James Moreland Johnson was born in Owensboro, Ky., on July 11, 1857, a son of Alfred B. and Martha (Moreland) Johnson. After receiving his preliminary education in his native town, he entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated with the degree of Civil Engineer in 1879.

Immediately after his graduation, Mr. Johnson entered the Government service and was stationed at Omaha, Nebr., and Council Bluffs, Iowa, as Transitman on the improvement of the Missouri River. In 1880, he returned to Kentucky and, until March, 1881, was engaged in private practice as

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[1] Memoir prepared by Charles H. Blackman and Woolsey M. Caye, Members, Am. Soc. C. E.

Engineer and Contractor at Owensboro. He was appointed Assistant Engineer for the Henderson Bridge Company on the construction of a bridge over the Ohio River at Henderson, Ky., and after the completion of that structure in March, 1882, he became an Assistant Engineer for the Louisville Bridge and Iron Company.

In March, 1883, Mr. Johnson entered the service of the Louisville and Nashville Railroad Company as Assistant Engineer in the Maintenance-of-Way Department where he remained until October, 1885. He then returned to the Louisville Bridge and Iron Company as Principal Assistant Engineer. In January, 1888, he was appointed Chief Engineer of the Bridge Company and in March, 1891, he was made President, which position he held until March, 1914. During that period Mr. Johnson was Consulting Engineer in charge of the construction of the Kentucky and Indiana Terminal Railroad Bridge over the Ohio River, at Louisville, Ky.

In 1914, he opened an office as Consulting Engineer, in Louisville, and continued in that business until his death. He served as Consulting Engineer for the Nashville, Chattanooga, and St. Louis Railway Company, the Illinois Central Railroad Company, the Louisville, Henderson and St. Louis Railway Company prior to its absorption by the Louisville and Nashville, the Central of Georgia Railway Company, the Mississippi Central Railroad Company, the Wabash Railway Company, and the Tennessee Central Railway Company. Included in his work for the Illinois Central Railroad Company was the strengthening of that Company's bridge over the Ohio River, at Cairo, Ill. In 1927, Mr. Johnson was employed by the Terminal Railroad Association of St. Louis, Mo., to review the Eads Bridge over the Mississippi River. He also made certain studies for the Missouri Pacific Railway Company, and designed a considerable number of highway bridges, and other structures. In March, 1932, he was again elected President of the Louisville Bridge and Iron Company, and just six days before his death had been appointed Building Inspector for the City of Louisville.

He was a Charter Member of the Engineers and Architects Club of Louisville, which was organized in 1891, and served as President in 1894 and 1909. He was elected an Honorary Member of the Club in 1934. Also, he held memberships in the American Railway Engineering Association and in the Pendennis Club, of Louisville.

By his professional associates James Moreland Johnson was held in the highest esteem, and his personal acquaintance was large, his friendships including men in all walks of life by reason of his pleasing personality and his many sterling qualities as a man.

In 1883, he was married to Ella Hicks, of Owensboro, Ky., who survives him. He is also survived by a daughter, Mrs. Carter L. Wilson, of Nashville, Tenn., and two grandsons, Lindsay and James Moreland Wilson.

Mr. Johnson was elected a Member of the American Society of Civil Engineers on July 1, 1891. He served as a Director of the Society from 1906 to 1908, and had been Chairman of the Local Membership Committee for a number of years.

FRANK HALL JOYNER, M. Am. Soc. C. E.[1]

DIED APRIL 11, 1933

The death of Frank Hall Joyner, at his home in San Marino, Calif., takes from the community one of the pioneer road builders of this part of California as well as an outstanding figure in the Highway Engineering field of the Pacific Coast.

Frank Hall Joyner, the son of Loomis M. and Mary (Cross) Joyner, was born at North Egremont, Mass., on January 20, 1862. He was a direct descendant of Robert Loomis, a Revolutionary soldier and of Joseph Loomis who was located at Windsor, Conn., in 1639. He received his early education in the public schools of his home town, and attended the Massachusetts Agricultural College, at Amherst, Mass., for three years, taking a special course in Mathematics.

In August, 1881, Mr. Joyner entered the employ of the New York, West Shore and Buffalo Railway Company, and served as Chainman, Rodman, and Assistant Engineer, respectively, holding the last position until the completion of the road in March, 1883. As Assistant Engineer he had charge under the Resident Engineer of considerable bridge and station work and the erection of the shops at Frankfort, N. Y.

Mr. Joyner was appointed Resident Engineer, in 1884, by the Wisconsin Central Railway Company and had charge, under the Chief Engineer, of the construction of nine miles of line. In July, 1886, he accepted a position with the Fitzgerald and Mallory Construction Company on the construction of the Denver, Memphis and Atlantic Railway, in Kansas. At the time of his resignation in 1887, he was Assistant Superintendent of Construction. He then entered the employ of the firm of Morison and Corthell, of Chicago, Ill., and was with this firm until 1891. In this position, Mr. Joyner had charge of the stone cutting for the bridge over the Ohio River, at Cairo, Ill., the Merchants Bridge over the Mississippi River, at St. Louis, Mo., the bridge over the Mississippi River, at Memphis, Tenn., as well as a number of bridges in and around Chicago.

Mr. Joyner then returned to the East and in 1892 entered the employ of the Pejepscot Paper Company, of Brunswick, Me., as Assistant Engineer in charge, under the Resident Engineer, of the erection of a pulp mill, boarding houses, etc., remaining with this Company until the completion of the plant in 1895.

In July, 1896, Mr. Joyner began the work in Highway Engineering in which he continued until his death. At that time he was appointed Resident Engineer by the Massachusetts Highway Commission in which position he continued until April, 1898, when he was made Division Engineer. He remained with the Massachusetts Highway Commission until 1911 when he resigned, to accept a position with the County of Los Angeles, California.

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[1] Memoir prepared by Earle A. Burt, M. Am. Soc. C. E.

On account of the notable ability Mr. Joyner had shown as a road builder in Massachusetts, his services were sought by the Los Angeles County Highway Commission to assist in the completion of its "Good Roads System" which was then being developed through a $3 500 000 bond issue.

As Chief Engineer for the Commission, Mr. Joyner served until the latter part of 1911, at which time the Los Angeles County Road Department was organized under his direction as Road Commissioner. During the period from 1911 until 1919, his services as Road Commissioner were continued, and many notable features in the development of the present highway system were accomplished. The necessity for trained highway engineers was particularly evident at this time, and Mr. Joyner's ability and success in developing and training competent engineers for this type of work was especially outstanding.

New developments in materials and methods led Mr. Joyner to resign from the County service and devote the next four years to consulting work with various organizations allied with road development. During this period, he was engaged on considerable work for the California Highway Commission. In 1924 he again became associated with the Los Angeles County Road Department as Consultant, and continued in this capacity until his death.

Mr. Joyner was married on October 4, 1888, to Clara Curtiss, of Brooklyn, N. Y., and is survived by his widow and his daughter, Mary Cross Joyner, a member of the Staff of the Mt. Wilson Observatory.

Mr. Joyner was elected a Member of the American Society of Civil Engineers on September 5, 1911. He was also a member of the American Association of Engineers.

EARL WALLACE KELLY, M. Am. Soc. C. E.[1]

Died January 30, 1934

Earl Wallace Kelly, the son of William Alexander and Laura Jane (Rollins) Kelly, was born on December 10, 1879, at Tawas, Mich. He was one of a family of six, of whom two brothers survive—Dr. Benjamin William Kelly, of Aitkin, Minn., and Carroll Ebert Kelly, a Contractor, of Burlington, Wash.

He was graduated from the Central High School, of Duluth, Minn., in 1896, and spent the following six years teaching school and working as Chainman, Rodman, and Deputy County Surveyor in Aitkin County, Minnesota. He entered the University of Minnesota in 1903 and was graduated therefrom with the degree of Civil Engineer in 1907. While at the University, Mr. Kelly's vacations were spent on surveys for the Milwaukee Railroad. After graduation, he was employed by the Duluth, Missabe and Northern Railroad and the Duluth and Iron Range Railroad Companies, and as Engineer for a general contractor on sewerage systems and street paving.

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[1] Memoir prepared by Felix Seligman, Supt. of Pumping Stations, City Water and Light Dept., Duluth, Minn., and T. F. McGilvray, M. Am. Soc. C. E.

In 1910, Mr. Kelly entered the service of the City of Duluth, as Assistant City Engineer, in charge of paving and sewers. In 1911, he was appointed Engineer of the Water and Light Department of the City of Duluth, and held this position until he resigned in September, 1917, to join the United States Army Engineer Corps. He went overseas as a Captain and served with distinction until he was discharged in August, 1919.

On his return from overseas, Mr. Kelly was engaged in private engineering practice with offices in Duluth, and did much efficient and highly satisfactory work for his clients. In 1926, he again entered the employ of the City of Duluth in the capacity of Engineer of the Water and Light Department; he became Manager of this Department in 1929, which office he held until his death.

Mr. Kelly was recognized in the city in which he lived, by the public, as well as by the Engineering Profession, as a wonderful organizer, an able executive, and a large-hearted and patriotic citizen, and he was greatly admired by those with whom he came in contact. His untiring efforts to better the Department he managed are evidenced by his rehabilitation of the Gas Distribution System, his construction of four new water reservoirs, his work of valuation of the Duluth Water and Gas Plants, and his last effort—that of remodeling the Lakewood Pumping Station, which stands as a tribute to his engineering skill and forethought.

He was a member of the Sigma Xi and Tau Beta Pi Fraternities, American Concrete Institute, American Water Works Association, New England Water Works Association, and American Gas Association, in all of which he took an active interest by contributing orally and in writing to current discussions and by holding numerous offices. Mr. Kelly was also a Mason, a member of the American Legion, the Duluth Engineers Club, and Order of Scottish Clan.

He was married on June 18, 1908, to Alice Jean Stewart, of Duluth, and is survived by his widow, a son, Miles William, and two daughters, Katherine Jean and Eileen Laura.

Mr. Kelly was elected an Associate Member of the American Society of Civil Engineers on August 31, 1915, and a Member on November 26, 1923.

EUGENE ASHBEL LANDON, M. Am. Soc. C. E.[1]

Died March 23, 1933

Eugene Ashbel Landon was born on May 22, 1855, in Salisbury, Conn., the son of Ashbel Landon and Laura B. (Wolcott) Landon. The history of Salisbury shows that the Landon family were in the town before the Revolutionary War and records that a certain Captain James Landon did the town credit in the 1755 Campaign against Crown Point in the French and

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[1] Memoir prepared by J. B. French, M. Am. Soc. C. E., from information supplied by Fred D. Rhodes, Esq., Groton, N. Y.

Indian War. His mother was a lineal descendant of Oliver Wolcott, member of the Continental Congress and signer of the Declaration of Independence.

In August, 1861, in the first year of the Civil War, his father enlisted in a Connecticut Regiment and was in active service until September, 1864.

During the Civil War the family moved to Vineland, N. J., and it was there that young Eugene received his early education. He was graduated from the Vineland schools, taught school for a time in that locality, and then entered Cornell University, at Ithaca, N. Y., where he took the course in Civil Engineering and was graduated with the Class of 1880.

Shortly after his graduation from Cornell, Mr. Landon obtained a position in the Maintenance-of-Way Department of the New York and New England Railway Company, and remained in that position until the spring of 1881 when he became Engineer for the Groton Iron Bridge Company, at Groton, N. Y.

In 1887, that Company was re-organized, took the name of The Groton Bridge and Manufacturing Company, and made Mr. Landon its Chief Engineer. He retained that position until 1901 when the Groton Company was included in the extensive merger of bridge companies which became the American Bridge Company which, in turn, shortly afterward became a subsidiary of the larger consolidation formed under the name of the United States Steel Corporation. After these consolidations the Groton Plant continued in operation for a year or two under Mr. Landon as Engineer and Manager, but it was then closed and his services were terminated.

Shortly after this, Mr. Landon opened a general store in Groton and combined its management with the cultivation of a small farm from which he obtained both diversion and profit by raising small fruits and vegetables, making the farm and the store effectively supplement each other.

While the closing of the Bridge Plant, with which he had been so long identified, practically ended his professional work, Mr. Landon always retained an active interest in engineering matters, and his engineering training and experience, which he had always put at the service of the community, were probably of greater benefit to the village and township after his professional retirement than before, because he was then able to devote more of his time to community matters.

Throughout the fifty odd years which Mr. Landon spent in the community, he was always interested in public affairs, serving at various times on the School Board, and other local boards. The variety and extent of his services are attested by the fact that, at the time of his death, he was Village Treasurer, a Justice of the Peace, a member of the Town Board, and Emergency Relief Officer for the town.

He was also a member of the Farm Bureau and its Local Representative. He was a member of the local Lodges of the Knights of Pythias and of the Masonic Order.

Mr. Landon was a man of kindly and genial character who won and held the regard and affection of those who worked under him, and the entire community of Groton held him in the highest esteem for his sterling character, his cheerful good fellowship, and his ever ready spirit of helpfulness.

Mr. Landon was married on March 26, 1884, to Mary Frances Rhodes, of Groton, N. Y., who died on May 14, 1931. They had no children but, in more cases than one, they took the equivalent of parental interest in the education and training of young people in their home. They were both always interested in the young people of the community and worked together in every way they knew to make the town a good and wholesome place in which to live.

Mr. Landon was elected a Member of the American Society of Civil Engineers on April 1, 1896.

OLIN HENRY LANDRETH, M. Am. Soc. C. E.[1]

DIED NOVEMBER 6, 1931

Olin Henry Landreth was born at Addison, N. Y., on July 21, 1852, the son of the Rev. James and Adelia (Comstock) Landreth. He received his early education in the High Schools, at Rushville, and at Penn Yan, N. Y., and at the Dundee and the Canisteo, N. Y., Academies. In 1876, he received, from Union College at Schenectady, N. Y., the degree of Civil Engineer and, in 1877, the degree of Bachelor of Arts. Later, in 1880, he was made a Master of Arts by Union College and, in 1908, he received from his Alma Mater the degree of Doctor of Science. He was a member of the Delta Upsilon Fraternity.

During school vacations, from 1870 to 1874, Mr. Landreth held various positions in railway engineering and during his college vacations, from 1874 to 1877, he assisted on surveys, investigations, and estimates of cost of waterworks at Schenectady, N. Y., and at Olean, N. Y. During 1876 and 1877 he served as Instructor of Physics at Union College.

In 1877, Mr. Landreth was appointed Assistant Astronomer of the Dudley Observatory, at Albany, N. Y., and remained in that position until 1879, when he was appointed Professor of Civil Engineering at Vanderbilt University, Nashville, Tenn. Professor Lewis Boss, Director of the Dudley Observatory, once said that astronomy lost a brilliant worker when Dr. Landreth assumed the responsibilities of family life and turned again to the practice and teaching of engineering.

After having served as Professor of Civil Engineering at Vanderbilt University from 1879 to 1886, he was made Dean of the Engineering Department at the University and continued in this position until 1894 when he was appointed Professor of Engineering at Union College.

Professor Landreth had a wide and varied experience in engineering practice which paralleled his teaching experience. While at Vanderbilt University he acted as Superintendent of Buildings and Grounds in charge of new construction and the maintenance and operation of the water supply, sewerage, and steam-heating systems. He also made surveys, estimates, and investigations for railroads and for water-works and electric light systems in various Southern cities. He was Consulting Engineer to the Board of Public Works of Nashville from 1883 to 1885; Consulting Engineer to the Davidson County, Tennessee, Bridge Commission, to the West Nashville Water Company, and to the Tennessee State Board of Health, making examinations and reports on smoke prevention, and on the sanitary effect of the construction of proposed locks and dams on the Cumberland River; and Special Commissioner to the World's Fair, Chicago, Ill., in 1893. He was a member of the Advisory Council of the Engineering Congress, at Chicago, in 1893; the New York State Water Storage Commission, in 1902 and 1903; the New York Bay Pollution Commission, 1903 to 1905; and of the Metropolitan Sewerage Commission of New York, 1906 to 1908.

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[1] Memoir prepared by H. Burdett Cleveland, M. Am. Soc. C. E.

Dr. Landreth was Professor of Civil Engineering at Union College from 1894 to 1919. During the twenty-five years of his service as Dean of the Engineering School, he built up the course and greatly modernized the teaching, bringing to the students the benefit of his direct contacts in his wide professional practice to supplement their theoretical training.

At the time of the Fiftieth Reunion of his Class in 1926 an endowment was presented to the College to further the teaching of Engineering at Union as a testimonial to Dr. Landreth's service, the funds having been subscribed by his former students. An embossed booklet was presented to him on this occasion which contained the following inscription:

"As a token of love and esteem to Olin Henry Landreth, C.E., A.B., Sc.D., of the class of 1876 at Union College, and in grateful recognition of his devoted and inspiring service as Professor of Engineering at Union College from 1894 to 1917, his former students have this day, on the occasion of the Fiftieth Reunion of his Class, presented to the college a sum of money for the purpose of establishing the Olin H. Landreth Engineering Fund, the income from which shall be applied, at the discretion of the Trustees, to the advancement of engineering at Union College."

When the United States entered the World War, Dr. Landreth obtained a leave of absence from Union College and served in an advisory capacity as Mechanical Engineer in the Ordnance Department, at Washington, D. C.

At the close of the World War, in 1918, Dr. Landreth resigned from his professorship at Union College and from that time carried on a consulting engineering practice with offices in New York City. He maintained his keen interest in the College; he aided in its advancement, and was always ready to confer and advise with his former students. His memory of men and of events was remarkable. It always gave him pleasure to meet a former student, to call him by name, and to refer to the work on which the man had been engaged since leaving college. His mind seemed to catalogue, in detail, every contact he had ever made.

His consulting practice was very wide and covered Mechanical Engineering as well as Civil Engineering, with most of his attention given to Sanitary Engineering. Among his later engagements was his work as Consultant over a term of years to the Eastern Potash Manufacturers Association and a review of an extensive engineering undertaking in Poland for American financial interests.

Dr. Landreth was a member of the American Society of Mechanical Engineers (Past Vice-President); the American Institute of Consulting Engineers (Member of Council, 1923 to 1926); American Society for Testing Materials (Past-President); Society for the Promotion of Engineering Education (Past-Director); Engineering Association of the South (Past-Secretary); Society of Engineers of Eastern New York; Schenectady Board of Trade (former Director); Special Joint Committee of Architectural and Engineering Societies on Structural Safety; Committee on Sanitation, Public Health, and Water Supply, of the Merchants Association of New York; American Association for the Advancement of Science (member of its Committees on Waterways and Water Storage and on Sanitation and Public Health); Sigma Xi Fraternity; and of the University Club of Mount Vernon, N. Y., where he had resided since 1919. He was a Republican in politics, and a member of the New York State Constitutional Convention of 1915.

Dr. Landreth was the author of "Instructions for Finding the True Meridian, with Tables for Surveyors," and "Metric Conversion Tables for Engineers." He was a lifelong member of the Methodist Episcopal Church.

In 1879, at Canisteo, N. Y., he was married to Eliza Taylor, a singularly sweet and gracious lady who endeared herself to all undergraduates who knew her at Union College in later years. Their children are: William Comstock (deceased), Olin Henry, Jr. (deceased), Mary Eliza (Mrs. C. S. Parker), Helen Adelia, James Taylor (deceased), and Robert Nelson.

Olin Henry Landreth was pre-eminently a teacher, of exceptionally broad mental balance in his conception of the functions of a teacher, and thorough to a marked degree in his own preparation for his work and in his guidance of students under his charge. His habits of mind were keen and searching. The principal characteristic of his work in the classroom, in any special investigation which he undertook, and in his professional practice, was the broad, deep foundation he laid to form the basis for his conclusions. His mind was unusually active, and he never hesitated to engage in, and to master, any new problem incident to the many professional demands made upon him.

His bent of mind was, also, essentially judicial and impartial, and some of his most important work was in connection with engineering Court cases. In discussing such problems he would say: "I am not a lawyer, but" and then proceed to a keen, legal, and complete analysis of the point at issue.

Dr. Landreth's manner in Court and in the classroom was austere. In contacts with his students outside the classroom and with those who knew him well in professional relationships, he was genial and kind, and disclosed an engaging sense of humor.

His outstanding attributes were a fine sense of honor, a courage which never knew defeat, and a faithfulness, enduring to the last days of his life, to the exacting labor of his profession, and to every stewardship which he had assumed.

Dr. Landreth was elected a Member of the American Society of Civil Engineers on September 3, 1884. He served as a Director of the Society from 1893 to 1895.

WARREN ALLSTON LELAND, M. Am. Soc. C. E.[1]

Died August 28, 1933

Warren Allston Leland was born on January 25, 1866, at Greenwood, S. C., where his father was in charge of a private school. He was the son of John A. and Ann Allston (Du Pre) Leland, and came of a prominent Charleston, S. C., family. His grandfather, Dr. Aaron Whitney Leland, was Pastor of the Scotch Presbyterian Church, of Charleston, and one of the founders of the Columbia Theological Seminary. His father, John A. Leland, was Professor of Mathematics at The Citadel (The South Carolina Military Academy) from 1845 to 1853; at Davidson College from 1854 to 1859; and in command of State troops, with the rank of Major in the Confederate Army, during the Civil War. His mother, Ann Allston Du Pre, was born in Christ Church Parish in South Carolina.

Warren Allston Leland received his preliminary education in Greenwood, and when The Citadel was re-opened in 1882, he matriculated as a cadet in the first class which entered that institution. He was one of the pupils of the late William Cain, M. Am. Soc. C. E., who was then a teacher there. He was graduated from The Citadel in 1886, with the degree of Bachelor of Science and, later, received the degree of Civil Engineer from his Alma Mater.

After his graduation, Mr. Leland entered the service of the Central of Georgia Railway Company and was engaged as follows: Rodman on surveys of the line from Carrollton, Ga., to Decatur, Ala.; Transitman on location and Resident Engineer on construction from Clayton to Ozark, Ala.; and Transitman on location and Resident Engineer during grading and afterward in charge of track-laying and bridge building from Columbus to Buena Vista, Ga. From 1889 to 1891, he was Resident Engineer in charge of track-laying and bridge building for the Company in Georgia and Alabama.

In 1891, Mr. Leland entered the employ of the United States Corps of Engineers as Civilian Inspector and Assistant Engineer. As such, he had charge of the hydrographic surveys connected with the construction of the jetties across the bar to Charleston Harbor, and with the improvement of rivers in the District. He was also in charge of building fortifications for the protection of the harbors of Charleston and Port Royal, S. C., on Sullivan's Island and on St. Helena's Island. Mr. Leland was in charge of river surveys and of building the Chanoine wicket dam, near Columbia, S. C.

From 1905 to 1907, Mr. Leland was Resident Engineer and Construction Manager for the Southern Power Company. In this position he built one 40 000-hp hydro-electric plant at Great Falls, S. C., and began a similar plant in the same locality. On this work he provided quarters for about 1 000 men.

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[1] Memoir prepared from information on file at the Headquarters of the Society.

He entered the service of the Yadkin River Power Company in 1907 as Resident Manager and, later, as Chief Engineer on the development of a 40 000-hp hydro-electric plant on the Peedee River, near Rockingham, S. C.

From 1913 to 1918, Mr. Leland served as President of the Tennessee Eastern Electric Company of Johnson City, Tenn. He constructed and operated for this Company a 10 000-hp hydro-electric plant on the Nolachucky River, near Greenville, Tenn.

Mr. Leland was a member of the firm of the Charleston Engineering and Contracting Company, from 1920 to 1922. This firm designed and constructed the "Greater" Citadel, and Mr. Leland took an active part in this expansion of his Alma Mater, which cost approximately $1 000 000.

In 1922 he became Vice-President and Chief Engineer of the Carolina Engineering Company, of Johnson City, Tenn., with which he was actively connected until his retirement from active business in 1928.

Mr. Leland was a member of the Presbyterian Church and of Etiwan Lodge No. 95, A. F. and A. M., of Mount Pleasant, S. C.

He was married in 1890 to Effie Drake Williams, of Clayton, Ala., and is survived by his widow, two sons, Warren Allston, Jr., and John A. Leland, and two daughters, Mrs. E. T. McKeithen, of Aberdeen, N. C., and Effie Leland.

Throughout his professional career, Mr. Leland was connected with large and important construction works. On these works he left the impress of his strong clean personality, and they stamp him as a master planner and builder and place him among the foremost civil engineers of the South.

Mr. Leland was elected a Member of the American Society of Civil Engineers on October 3, 1900.

ALFRED FELLOWS MASURY, M. Am. Soc. C. E.[1]

DIED APRIL 4, 1933

Alfred Fellows Masury was born in Danvers, Mass., on September 2, 1882, the son and only child of Charles H. Masury and Evelyn (Fellows) Masury who were direct descendants of Governor John Endicott of Massachusetts. His father served as a Captain in the Civil War; was wounded at the Battle of Bull Run; and took an active part in civic affairs up to the time of his death. His mother was generously endowed with traditional New England characteristics, and has always taken an active part in National, local, civic, and political organizations, such as the Daughters of the American Revolution, the National Women's Suffragette Movement, and local Republican organizations.

Alfred Fellows Masury attended the public schools of Danvers, and was graduated from the local High School. He then attended Brown University, at Providence, R. I., from which he was graduated with the degree of Mechanical Engineer in 1904. During his summer vacations while at school, he worked in various capacities with industrial concerns in and around Providence, and other New England points, gleaning a foundation of practical knowledge which served him in good stead throughout his business career.

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[1] Memoir prepared by J. F. Winchester, Esq., Newark, N. J.

After graduating from Brown University, Mr. Masury became connected with the General Electric Company, of Lynn, Mass., and served that concern from 1904 to 1906 as a Junior Engineer. With this background of experience he became associated with the Vaughn Machine Company, of Peabody, Mass., and carried forward various types of design which related to the construction of leather working machinery, foundry apparatus, cotton machinery, machinery tools, printing machinery, etc.

He realized early that family background meant little to him if he was to make a mark in the world; that his having been reared as an only child may have established certain selfish traits that might act as a handicap to him. The balance he established in this respect won him friends in all walks of life and created for him an opportunity to live a full useful life.

When the Vaughn Machine Company began to experiment with various types of automotive equipment and started to build, under contract, internal combustion engines for various companies, Mr. Masury's interest was attracted and, at that time, he visualized the future of the automotive industry and attached himself to it by specializing in the various designs on which this Company was engaged. During this period he rendered assistance to Mr. Ralph Hood, of Danvers, which established these two men as pioneers in the advancement of the self-starting automobile. Their creation—the Gasaulec—was displayed at an early Automobile Show held at Boston, Mass. Like many early inventions, it was ahead of its time, and public reaction was not favorable to it. Ultimate financial difficulties were encountered, and the project was discontinued as an established machine.

The Vaughn Machine Company was also engaged in the construction of a number of steam trucks, and Mr. Masury played an important part in their design and construction. This was at a period when rock maplewood blocks proved to be better tires than the rubber ones of that day.

In November, 1907, he was employed by the Hewitt Motor Company as Engineer, and assisted Mr. E. R. Hewitt in developing that Company's line of equipment. He designed the Hewitt 3, 5, 7, and 10-ton chain-drive trucks and supervised their construction. At this time, Mr. Hewitt and he became acquainted with Mr. George Saybolt, of the Standard Oil Company of New Jersey, and co-operatively they designed, and Mr. Masury superintended the construction of, the first road-oiling and hot-pressure penetration distributor for hot asphalt binding of macadam roads. These were the first motorized units of this kind in the world.

Mr. Masury also promoted and designed highway-sprinkling devices; automatic tail-gate mechanism for spreading stone on highways; special bodies and trucks for mixed asphalt delivery and spreading, and special equipment for distributing wet mixed concrete from the end of a truck. It was during this period that he became very much interested in highway construction and took a special course in this subject at Columbia University, in New

York City. Through this course, he contacted with many men engaged in civil engineering pursuits and felt that close contact could be made with them through joining the Society.

When the Hewitt Motor Company was absorbed in the consolidation of the Mack, Hewitt, and Sauer Trucks, Mr. Masury became Manager of the New York Service Station, and afterward was promoted to the position of Chief Engineer of the International Motor Company. From 1914 until his death, he was also a Vice-President and Director in charge of operations of Mack Trucks engineering service and experimental shops, developing trucks, buses, and rail cars. He was responsible for the design and development of the Mack "AC" truck, for its combination dash and radiator, mid-ship transmission with single-shift rod control, and many of the other various designs that entered into the construction of this unit.

At about this time, he was married to Edna Lanphear, of Providence, R. I., whose practical knowledge of engineering matters and business ability had established her as Office Manager in the New York Office of one of the largest manufacturers of roller bearings. Her congenial nature, combined with her practical knowledge of engineering, and her ever constant application to Mr. Masury's welfare, without a doubt, promoted materially the possibility of his covering so successfully such a wide line of endeavors.

During the World War, the Corps of Engineers of the United States Army adopted the Mack "AC" model as its standard equipment, and, in this connection, Mr. Masury was responsible for the "AC" search-light trucks with direct engine-driven generators for the Army Anti-Aircraft Corps; the anti-aircraft crane truck; and the anti-aircraft gun-mount trailer.

He was also responsible for the Mack Company's full line of passenger buses and worked on the many special features for which they are known. He also developed the Mack gas-electric driven, six-wheel buses, with power-dividing drive to the two rear axles. Many minor individual inventions and designs are to Mr. Masury's credit; but outstanding among these was his early realization of the value of rubber for shock and vibration absorption. He was the first to promote its use in the automotive industry for this purpose, and his early design of rubber, shock-insulated, spring mounting led this industry to give careful consideration to the possibilities of this material. He further applied the use of rubber to the rubber-rimmed steering wheel; rubber shock-insulated transmission and engine mounting; rubber torque-insulated drive shaft, etc.

Mr. Masury also made special studies of the application of the internal combustion engine to rail cars, and had an intimate knowledge of the problems relating to railway passenger and freight transportation. One of his most recent experiments was in the field of truck body refrigeration which resulted in the development and establishment of the Mack "Statothern" refrigerated body for this class of vehicle.

When the U. S. Corps of Engineers took over the output of the Mack "AC" trucks for the duration of the World War, Mr. Masury was drafted to carry on the design. In addition, he served as an Automotive Technical

Adviser in the development of standardized Army trucks for the Quartermaster Corps and helped to create the U. S. Army "Liberty" truck.

As a result of this experience, he never lost interest in the welfare of his country in relation to this branch of service. The Society of Automotive Engineers appointed him to the position of Chairman of the Army Ordnance Advisory Committee, which position he held from 1928 to the time of his death. During this period he rendered outstanding service as attested by Maj.-Gen. Samuel Hof, Chief of Ordnance, U. S. Army, in a letter to John A. C. Warner, Manager of the Society of Automotive Engineers, Incorporated, as follows:

> "Colonel Masury's work was of the highest order and he gave himself unselfishly and unrelentingly to the problems which were presented to him. His always sound judgment and engineering knowledge was an asset upon which we greatly relied."

and from Lt-Col. C. M. Wesson, of the Ordnance Department, with whom Mr. Masury was more intimate, comes the following:

> "Colonel Masury was an enthusiastic advocate of national defense and gave unsparingly of his time and abilities to this work in co-operation with the Ordnance Department of the Army. He was a charter member of the Army Ordnance Association, a Lieutenant Colonel in the Ordnance Reserve and, at the time of his death, was chairman of the Ordnance Advisory Committee of the Society of Automotive Engineers, in which capacity he had served for the last five years. In this capacity, he rendered notable service and approached the problems presented without fear, favor, or prejudice, and with the sole idea of guiding developments along sound lines so that his country should, if possible, have the best equipment obtainable."

As a member of the Military Motor Transport Advisory Committee of the Society of Automotive Engineers, Mr. Masury played an important part in the development by the Quartermaster Corps, of the first mechanized company to displace cavalry. In the first review of that force held at Fort Eustis, Virginia, about 1931, he remarked:

> "It is regrettable that the average civilian and politician do not recognize the full importance of the advances made in motor transport since the World War. It handicaps our preparedness and imposes an unnecessary hardship upon our Army personnel who have to struggle along and experiment with designs of the World War period."

In 1923, Mr. Masury was Second Vice-President of the Society of Automotive Engineers, representing motor-car engineering. In 1925, he was Vice-Chairman of the Metropolitan Section. He also served on the following committees: Diesel Engine Activity Committee; Automotive Transport Code Committee; Military Motor Transport Advisory Committee; Motor Coach and Motor Truck Activity Committee; and Transportation and Maintenance Activity Committee. He was Chairman of the National Membership Committee for 1925 and, in the same year, served as Vice-Chairman of the Motor Coach Division of the Standards Committee.

He took an active part in the co-operative motor-truck impact tests conducted by the U. S. Bureau of Public Roads and made an outstanding contribution to the subject by conducting—on November 19, 1919, at 79th Street and the North River, New York City—a series of original truck "jump" tests which definitely established many facts on this subject, and which have since served as a guide to automotive and highway engineers.[2] This established Mr. Masury as the first automotive engineer to use the motion picture camera for the study of relative action of road impact, spring and tire deflection, and action of load in relation to the vehicle.

It can be readily seen from these experiences that such activities brought Mr. Masury into contact with many of the world's outstanding engineers and engineering projects. He never failed to contribute personally to their welfare when it was possible for him to do so. As a result, his versatile mind and talents broadened with time.

The following engineering organizations of which he was a member, testify to his many various activities: Society of Automotive Engineers; American Society of Mechanical Engineers; Institute of Automobile Engineers (England); American Institute of Electrical Engineers; Permanent International Association of Road Congresses (Paris); Society of American Military Engineers; American Society of Municipal Engineers; United States Naval Institute (Associate Member); Sons of Union Veterans of Civil War; New York State Society of Professional Engineers and Land Surveyors; New York State Board of Licensing for Professional Engineers; American Association for the Advancement of Science; American Petroleum Institute; American Road Builders Association; Army Ordnance Association; Institute of Metals; and the Royal Society for the Encouragement of Arts, Manufactures, and Commerce, of London, England. He was other than a "joiner." His association with an organization was an implied tribute to its work and its efficiency. He had many friends in the petroleum industry and took a particularly keen interest in the activities of the American Petroleum Institute.

It was only natural that Mr. Masury's close association with the U. S. Army should lead him to be intrigued with the possibilities of aviation. He was an ardent advocate of aviation and used it for transport purposes. Lighter-than-air craft seemed to appeal particularly to his inquisitive mind. Mr. Hugh Allen, of the Public Relations Department of the Goodyear Tire and Rubber Company, of Dayton, Ohio, states that when the *Graf Zeppelin* first visited this country, an invitation was received from Dr. Hugo Eckener and arrangements were made for Mr. Masury to take a flight on it; but unfavorable weather conditions prevented the flight being made. Later, he made a flight on the U. S. S. *Shenandoah* and on May 17, 1927, as guest of Dr. Eckener, he started to make the flight from Europe to the United States on the *Graf Zeppelin.* The ship was buffeted around Europe for two or three days and was finally turned back, because of the failure of four of the five motors aboard. Dr. Eckener was able to navigate the ship back to a safe landing. Mr. Masury, however, undaunted always felt that this type of unit presented certain possibilities that were well worth while. Truly a pioneer, he never shirked his duty when he thought a contribution could be made to the advancement of science. His experiences aboard various types of aircraft impressed upon him and acquainted him with the hazards of aviation, particularly lighter-than-air craft; yet the pleadings of his friends to pursue less hazardous studies, remained unheeded because of the Viking spirit he possessed. At the time of his death, he had probably traveled upon a greater number of air-transport types of equipment than any other civilian.

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[2] *Transactions,* Soc. of Automotive Engrs., Vol. 15, 1920, Pt. 2.

Mr. Masury's activities of a public nature, particularly that which had to do with the use of automotive equipment, resulted in his taking a very active interest in the construction of the Holland Tunnel. While he did not serve directly as a member of the Committee, he recommended to the late Clifford M. Holland, M. Am. Soc. C. E., Chief Engineer of that organization, that Mr. H. K. Moran, one of his engineering assistants, and the writer, be appointed to an Advisory Transportation Engineering Committee which Mr. Holland was about to select, to study the approaches and exits of the tunnel. This Committee's activities are a part of the Annual Report of the Tunnel Commission covering the functions for the year 1920. Mr. Masury frequently consulted about this work and took an active interest in all problems relating to it. He was particularly interested in and studied thoroughly the ventilation problems relating to the tunnel. Henry M. Crane, Past-President of the Society of Automotive Engineers, testifies that Mr. Masury and he joined in a letter to Mr. Holland, suggesting ways and means for a proper system of ventilation for the tunnel.

Mr. Masury's contacts as a Sales Engineer led him into many State and municipal departments, where he gave freely of his advice and experience to promote the economical transportation of material. In this connection, he played an important part in advising contractors who were buying his equipment for the hauling of material at the Boulder Dam. He had made a number of visits to that project, and was greatly impressed with its possibilities.

The list of his clubs as noted, distinctly indicates his choice of recreational activities: New York Athletic Club; North American Yacht Racing Association; Port Washington Yacht Club; Cruising Club of America; Montauk Yacht Club; and Lloyds Harbor Club (Commodore). He was also a member of Sigma Chapter of Psi Upsilon Fraternity, New York, N. Y.

His attendance at track meets, golf, boxing, and racing affairs, testified further to his love of sports. He was tolerant toward all of them; but his greatest recreation came from his association with his principal hobby—boating. In his first boat, a Gloucester dory, fitted with a kerosene engine, he derived great pleasure on the waters surrounding the rugged New England shores; and, on the last, *L'Apache,* a luxurious yacht frequently seen in the various waters around New York and vicinity, he found rest and happiness with his family and friends.

Mr. Masury was known to many yachtsmen of the New England Coast and Long Island Sound because of his active interest in boat-racing events of various types, and the valuable engineering advice he was able to render them on motor applications. He had personally served in the last few years in various official capacities on committees of the Bermuda Race, the Gibson

Island Race, and the Gold Cup Race, and had loaned his 70-ft cruiser, *L'Apache*, as the official committee boat for these events. Generous to a degree, he always shared his worldly possessions unstintingly with his friends and made his own convenience subservient to the wishes of those who sailed with him; thus, his many friends were always assured of a hearty welcome aboard his yacht.

Too frequently, in engineering circles, one is apt to judge his associates by their failures in one endeavor or another. That is a prevalent human fault. Mr. Masury was ever tolerant, generous, and sympathetic. If he ever strayed from the teachings of his noble mother, he promptly charted his course along the path of the fundamental thoughts which dominated his mind: "Have I been unfaithful in dealing with others? Have I been untrue to my friends? Do I practice what I preach? In learning and straightway practicing—is there not pleasure also? When friends gather around from afar, do we not rejoice? Whom lack of fame cannot vex, is not he a gentleman?"

Such were the thoughts that made Mr. Masury particularly devoted to his mother and created for him a happy family life; that enabled him to count among his friends the thousands whose names appear upon the rosters of the organizations to which he belonged; and that enabled him to associate most intimately with the world's outstanding scientists, industrialists, and Government officials who ever sought his advice and guidance.

He lived a full life—his sales ability and practical contact with everyday life enabled him to impress his associates with the current developments of his brain; many of them have been applied and are accepted as commonplace in the construction of automotive equipment which is working on the highways and byways of America and the world. Knowledge passed along to mankind by such pioneers, is unto itself a monument. Mr. Masury's contributions to the engineering world will be utilized for the benefit of humanity and the glory of his country long after many have been forgotten.

If a young engineer were to ask how it was possible for a man to have such outstanding accomplishments and yet pass along at the age of fifty years. it could only be stated that it was brought about through constant application and long arduous hours, adding to it the precepts that have been set forth by Mr. A. Griswold Herreshoff, who, in the *Journal* of the Society of Automotive Engineers, states:

> "Although the following maxims were not so expressed or written, they were the guiding spirit of A. F. Masury's engineering organization:
>
> "Always design the simplest and best way you know how, and use the best materials available.
>
> "Know what is being done in this country and in Europe in the same field.
>
> "Never adopt a design, only an idea or basic principle; but develop your own design, based on engineering reason and calculation.
>
> "Give an engineer the responsibility for a development and have him follow it through conception, design, testing, perfecting, and its production."

Mr. Masury was the only civilian aboard the U. S. S. *Akron* and perished with it off Barnegat Bay, New Jersey, on April 4, 1933. His untimely end in the wreck of the *Akron* brought grief to all his associates and intimate friends, but also gratification that he died a hero in the service of his country so that his many qualities and accomplishments will forever be enshrined in the hearts of his countrymen.

To conform to his oft-expressed wish for such a resting place, his ashes were taken from the Brooklyn Navy Yard on the U. S. Coast Guard Cutter *Champlain,* to the spot where his life was cut short. There, simple services were conducted, and his ashes were scattered to the four winds and the sea, with full faith that death for him was but the gateway to an eternity of opportunity.

His immediate survivors are his mother, Mrs. Evelyn Fellows Masury, and his widow, Mrs. Edna Lanphear Masury.

Mr. Masury was elected an Associate Member of the American Society of Civil Engineers on January 6, 1915, and a Member on October 10, 1927.

SIR ERNEST WILLIAM MOIR, M. Am. Soc. C. E.[1]

DIED JUNE 14, 1933

Ernest William Moir was born in London, England, on June 9, 1862, the son of the late A. M. and Isabella (Japp) Moir, of Montrose, Scotland. He was educated at University College School and at University College, London. He served his pupilage with A. Chaplin and Company, of Glasgow, Scotland, and afterward with R. Napier and Sons, Shipbuilders, of Glasgow, receiving practical experience in the engine shops and shipyard.

Later, he joined the staff of Tancred, Arrol, and Company, the builders of the Forth Bridge, and was in charge of the erection of the southern cantilevers, directly under the great bridge builder, Sir William Arrol. The late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., who designed the Forth Bridge, appointed Mr. Moir to act as his Resident Engineer on the Hudson and Manhattan Tunnels, in New York, N. Y. The contract for this work was later placed with S. Pearson and Son, of London, and after designing the shield, Mr. Moir became the Agent for S. Pearson and Son in charge of the work. When the operations were suspended on the Hudson and Manhattan Tunnels, due to the Baring financial crisis, he had driven 2 000 ft of cast-iron lined tunnel in less than a year, and had just reached the rock on the Manhattan side.

In 1891, Mr. Moir returned to England, and was entrusted with the construction of the Blackwell Tunnel beneath the River Thames, for S. Pearson and Son. He designed all the necessary plant and planned the carrying out of the work. Simultaneously, he supervised the construction of the Surrey Commercial Docks and, in 1900, became a Director of the firm of S. Pearson and Son. Other large contracts under his direction were: Great Northern and City Tube Railway, in London; Seaham Harbor, on the east coast of England; Admiralty Harbor, at Dover, England; four tunnels under the East River, at New York, for the Pennsylvania Railroad Company; Royal Albert Dock Extension; harbor works, Valparaiso, Chile; and Silent Valley Water-Works for the City of Belfast, Ireland.

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[1] Memoir prepared by Sir Henry Japp, M. Am. Soc. C. E.

Sir Ernest took an active part during the World War for the Ministry of Munitions, founding the Inventions Branch of this Ministry, in London. He later visited the United States on behalf of the British Government, and established a Government Department in New York, residing there from 1915 to 1916. Between 1915 and 1919, he served under four Ministers at the Ministry of Munitions. He was commissioned by Mr. Lloyd George to report on the production of shells in France. On his return from the United States, he was appointed Director-General of American Supplies, and was responsible for purchasing rails and rolling stock for the various Allied fronts. After the war, he acted as Chairman of the British Transport Liquidation Committee and disposed of the railways and equipment. He was President of the Inter-Allied Non-Ferrous Materials Committee. For these activities during the war, he was created a Baronet in 1916, and an Officer of the Legion of Honor in 1919.

Other governmental works consisted of membership in the Government Departmental Committee on Deep Excavations, giving evidence before the Royal Commission on Oil Fuel Storage, and the Admiralty Works Department relating to departmental organization and efficiency. Sir Ernest was also a member of the Admiralty Engineering Committee and the Admiralty Civil Engineering Committee for some years, and before and during the war reviewed proposed modifications and improvements in the Government plans for naval bases.

Sir Ernest was a man of great energy and driving force, and very prolific of practical ideas for the carrying out of engineering works. Among the many devices which he contrived, none is better known than the medical air-lock, which he first used on the construction of the old Hudson and Manhattan Tunnels in 1890. The medical air-lock, in which workmen suffering from caisson disease may be recompressed in order that the nitrogen bubbles may be re-absorbed in the blood and tissues, is considered a necessary part of the equipment on all work under air pressure. The wonderful results obtained in overcoming caisson disease by this method has made Sir Ernest's name well-known throughout the Engineering World.

In 1887, he was married to Margaret Bruce, daughter of the late John Pennycock, of Ravelstone and Dalmeny, Midlothian, Scotland. He is survived by Lady Moir, and his only remaining son, Captain Arrol Moir.

He was a member of the Institution of Civil Engineers, in London, and a Fellow of the University College, London.

Sir Ernest was elected a Member of the American Society of Civil Engineers on September 7, 1904.

JEROME NEWMAN, M. Am. Soc. C. E.[1]

Died April 30, 1933

Jerome Newman was born in San Francisco, Calif., on April 16, 1862, the son of Edward and Johanna (Bendan) Newman. He was educated in the public schools of San Francisco, was graduated from the University of California, Berkeley, Calif., in 1883, with the degree of Bachelor of Science (Civil Engineering), and studied three years at the then new Royal Polytechnic School, at Charlottenburg, Prussia, subsequently known as the Technical University of Berlin, Germany.

Returning to California, Mr. Newman began his professional career as Field Draftsman and Transitman on railroad location for the Southern Pacific Company, under W. E. Marsh, Assistant Engineer, from Santa Ana to Escondido and from Burbank to Santa Barbara, Calif., the job lasting about ten months. The record of his subsequent assignments may be summarized as follows: In 1888, he was with the Central Irrigation District, in Colusa and Glenn Counties, California, as Levelman, under C. E. Grunsky, Past-President, Am. Soc. C. E., Chief Engineer. From 1889 to 1890, Mr. Newman was engaged on miscellaneous land surveys, etc.

From 1890 to 1893, he was with the Northern Pacific Railroad Company, first as Resident Engineer in charge of double-tracking from Puyallup to South Tacoma, Wash. (16 miles); and, later, as Resident Engineer on Maintenance of Way, on the Pacific Division from Seattle, Wash., to Portland, Ore., and the Rocky Mountain Division from Sand Point, Idaho, to Livingston, Mont., in charge of track work, bridges, and buildings, under A. Rydstrom and H. C. Relf, Division Engineers. In 1894, he served as Topographer for the proposed Overland Pacific Railroad, at Fort Bragg, California, a project that was subsequently abandoned.

From 1895 to 1900 Mr. Newman was engaged with The San Francisco and San Joaquin Valley Railway Company (now the Atchison, Topeka and Santa Fé Railway Company), from San Francisco to Bakersfield, Calif., as Bridge Designer under A. D. Schindler, M. Am. Soc. C. E., Chief Draftsman, and, afterward, as Chief Draftsman and Office Engineer under W. B. Storey, Jr., M. Am. Soc. C. E., Chief Engineer.

In 1900, he entered the service of the Southern Pacific Company as Principal Office Assistant Engineer, under the late William Hood, M. Am. Soc. C. E., Chief Engineer, and was engaged in estimating and reporting on projected and purchased lines and other engineering projects and in supervising the design of structures. In 1904 and 1905 he was with the San Pedro, Los Angeles and Salt Lake Railroad Company, at Los Angeles, Calif., as Office Engineer on Construction under E. G. Tilton, Chief Engineer, and during 1905 and 1906, he was again in the employ of the Southern Pacific Company, as Principal Office Assistant Engineer.

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[1] Memoir prepared by a Committee of the San Francisco Section, consisting of E. T. Thurston and A. D. Schindler, Members, Am. Soc. C. E.

In 1907, Mr. Newman was with the North Coast Railway Company (now part of the Oregon-Washington Railroad and Navigation Company), as Construction Engineer, from Wallula to North Yakima, Wash., and from 1907 to 1912, he again served the Southern Pacific Company as Principal Office Assistant Engineer. In 1912, Mr. Newman was appointed Chief Engineer in charge of design, construction, and maintenance of harbor facilities, at San Francisco, by the California Board of State Harbor Commissioners, which position he held until 1916. He then engaged, from 1916 to 1923, in private consulting practice, specializing in railroad and harbor work, chiefly as expert witness before the California Railroad Commission and the Interstate Commerce Commission and as Arbitrator in disputes between owners and contractors. Most important during this period was an engagement (1922–1926) as Consulting Engineer on the reconstruction of seventeen miles of the Yosemite Valley Railroad, necessitated by the construction of Exchequer Dam by the Merced Irrigation District, which railroad work cost more than $5 000 000.

Endowed with a keenly analytical mind, resourceful, and eminently practical, an unusually retentive memory for important facts, a fine sense of moral and ethical values, and always the courage of his convictions, Mr. Newman, notwithstanding a retiring disposition, built upon a broad educational foundation a professional career that brought him the respect and confidence of fellow engineers and clients alike. In the words of an engineer acquaintance of more than fifty years: "In character and ability he had everything that a good engineer should have."

Both the scientific and ethical sides of the Engineering Profession were his constant concern, and he was unselfishly interested in the welfare of his fellow engineers, particularly the younger men, many of whom will cherish the memory of his sympathetic, helpful friendliness. Although more inclined to study and quiet conference than to general discussion, he maintained a constant interest in Society affairs and served as President of the San Francisco Section during 1918. He was also an Associate Member of the American Railway Engineering Association, a member of the Pacific Association of Consulting Engineers, and a member of the California Academy of Sciences.

Mr. Newman's personal interest in the general public welfare was evidenced by active membership in the Commonwealth Club of California. Fraternally, he was a Scottish Rite Mason and a Shriner. Habitually, however, his evenings were spent in the family circle where his books and microscope and music afforded culture and recreation to his taste; and thus it was at home, on the evening of April 30, 1933, without forewarning, that he passed peacefully, painlessly away.

He was married in Tacoma, Wash., on September 24, 1892, to Louise J. Moore who, with their son and daughter, and his sister, survives him.

By his passing, the profession loses an able, honorable member, his community an intelligent, upright citizen, his family a devoted husband and father, and his associates a sympathetic, helpful, faithful friend; but the influence of his sterling character lingers to the everlasting benefit of all who had the privilege of knowing him.

Mr. Newman was elected a Member of the American Society of Civil Engineers on January 7, 1913.

FRED ADOLPH NOETZLI, M. Am. Soc. C. E.[1]

Died May 24, 1933

Fred Adolph Noetzli was born in Hoengg, near Zurich, Switzerland, on July 29, 1887. He was the son of Rudolf and Hermine (Gwalter) Noetzli. He came from old, substantial peasant stock, best described perhaps as Switzerland's truest aristocrats, the family records tracing back to 1440. He clearly showed in his early youth the love for Mother Earth and the respect for ownership of ground and soil.

After the completion of his elementary and High School education, Mr. Noetzli was summoned to military service in the Field Artillery, in the course of which he contracted an acute heart disease which was destined to restrict his physical exertions thereafter and to become the direct cause of the untimely end of a most successful and promising career.

In the fall of 1907, he entered the Federal Institute of Technology, in Zurich, as a student of civil engineering and was graduated with the diploma of this institution in 1911. In the fall of the same year, Mr. Noetzli became Assistant to Dr. Fritz Baeschlin, Professor of Geodesy at the Institute, and, simultaneously, began work on his thesis on "The Accuracy of Sighting with Telescopes," which he completed in the fall of 1913, and for which he was awarded the degree of Doctor of Technical Science.

From 1914 until the spring of 1915, Mr. Noetzli was Designing Engineer with the subsidiary of the well-known German firm, Wayss and Freytag, in Naples and Rome, Italy. Subsequently, he returned to Zurich to lecture for Professor Baeschlin during the latter's absence on military service.

Late in the fall of 1915, Mr. Noetzli came to the United States and after "breaking in" as a Draftsman in Baltimore, Md., became Instructor in the Civil Engineering Department, at Lehigh University, Bethlehem, Pa., where he remained until 1917, when he moved to Los Angeles, Calif., to work as a Structural Engineer. Not until 1918 did Mr. Noetzli enter the field of Hydraulic Engineering, and particularly the design of dams, which was to lead to his outstanding professional achievements.

In 1920, he was Chief Engineer of Beckman and Linden, of San Francisco, Calif., and it was under his direction that the design of dams, power stations, and canal lines of the Verde River Irrigation Development, in Arizona, was prepared which, in 1921, was approved by the United States Bureau of Reclamation.

After a brief trip to Switzerland in 1921, Mr. Noetzli established himself as a Consulting Hydraulic Engineer, in Los Angeles, and, in 1922, he became Secretary of The Engineering Foundation's Committee on Arch Dam Investigation, in which capacity he designed the test arch dam which was built in Stevenson Creek, in Southern California.

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[1] Memoir prepared by Paul Baumann, M. Am. Soc. C. E.

He subsequently became connected with the following projects: In 1923, the design and specifications for the Malibu (arch) Dam, in the Santa Monica Mountains, California, which was built in 1925; and the design of the Green Valley (multiple-arch) Dam, near Lake Arrowhead, California, in conjunction with Quinton, Code, and Hill, Consulting Engineers, built in 1924 and 1925; and in 1924 and 1925, raising Mountain Dell (multiple-arch) Dam, Salt Lake City, Utah, from a height of 105 to 150 ft.; from 1925, as Consultant for the United States Indian Irrigation Service on the Coolidge (multiple-dome) Dam and Power Development, on the Gila River, in Arizona; 1927 to 1931, Consultant for the Los Angeles County Flood Control District, on the Pacoima (constant-angle arch) Dam, 372 ft. high; the Big Dalton (multiple-arch) Dam, 165 ft. high; and the Sierra Madre, Sunset Canyon, and Hanson (arch) Dams. In 1927 and 1928 he was engaged on the design and construction of Railroad Canyon Dam, in Southern California, 100 ft. high; from 1928 to 1931, as Consultant for the Mexican Government on the design and construction of the Rodriguez Dam, on Tijuana River, Baja California; from 1928 to 1933, Consultant for the City of Pasadena, Calif., on the 370-ft. Pine Canyon (gravity) Dam, on the San Gabriel River, in California; a review of the designs of Santeetlah (arch) Dam, built in 1927-1929, in North Carolina, by the Aluminum Company of America; an improved type of Ambursen Dam for heights of 200 to 250 ft., for the Ambursen Construction Company, of New York, N. Y.; the Bananeiras (multiple-arch) Dam, in Brazil, built in 1929 and 1930 by the Electric Bond and Share Company of New York City; and reports to the Swedish Government, the City of São Paulo, Brazil, and the Tirso Hydroelectric Company, Italy, on the construction of arch and multiple-arch dams in the United States.

Mr. Noetzli was the author of several papers published by the Society and for the one on "Gravity and Arch Action in Curved Dams,"[2] was awarded the J. James R. Croes Gold Medal, in 1921. He wrote the chapter on "Multiple-Arch Dams" in the 1927 Edition of "The Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E., and also wrote the chapter on "Dams" in the new edition of the Encyclopedia Britannica, and many other articles on dams in the United States and abroad.

In August, 1916, Mr. Noetzli was married in New York, N. Y., to Klary Vogt, who survives him.

He was a member of the Turnerschaft Utonia, in Zurich, a dueling fraternity; the G. E. P., an Alumni Society of the Federal Institute of Technology, in Zurich; the American Concrete Institute; and The Engineering Foundation. He was a man of exemplary courtesy and self-control, with high ambitions and a keen, creative mind.

Mr. Noetzli was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919, and a Member on July 12, 1926.

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[2] *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

DWIGHT RAYMOND REDMAN, M. Am. Soc. C. E.[1]

DIED MAY 14, 1933

Dwight Raymond Redman, the son of William Harrison and Carolyn (Kinsley) Redman, was born at Columbus, Ind., on June 3, 1884.

He decided early in life that he would follow the profession of Civil Engineering and, at the age of sixteen, he was engaged in field survey work for the United States Indian Irrigation Service, in Yakima Valley, Washington. He served the Indian Service in various capacities from 1900 to 1905, acquiring knowledge that became useful to him during his college days, and also later in life. The progress made by Mr. Redman before he had had the benefit of an education, is shown by the fact that he was Chief of Party for the Indian Service from 1904 to 1905.

While his success had been more than ordinary prior to 1905, he realized that an education would be of benefit for a professional career. In 1905, he entered Washington State College at Pullman, Wash., enrolling in the Department of Civil Engineering. He pursued his studies assiduously, and was noted for the conscientious manner in which he went about his work. Other subjects that interested him, included business management and stage setting; during his college career, he held offices in several student bodies, requiring knowledge in both fields.

Upon graduation, Mr. Redman served as Location Engineer for the Toppenish, Simcoe, and Western Railroad, a branch of the Great Northern Railway. Following this, he entered private practice, maintaining his own office from 1910 to 1913. The work handled during this period consisted largely of municipal improvements for small cities in the State of Washington.

A service that extended for more than two decades was begun in 1913, when Mr. Redman was engaged by the United States Indian Irrigation Service to make a survey of irrigation possibilities on the Colville Indian Reservation. This work consumed two years, after which he was placed in charge of the construction of the Little Nespelem Project.

Upon the successful completion of this job, he was advanced to the position of Office and Designing Engineer on the Wapato Project, on the Yakima Reservation, in Washington. As a further reward for his services, Mr. Redman was made Project Engineer in April, 1918, in charge of the Wapato Project and three other smaller projects on the same Reservation. His new duties involved not only design and construction, but maintenance and operation of the projects as well. This position was held by him until he was transferred to the San Carlos Project, in Arizona, in March, 1932.

The projects on the Yakima Reservation included a total of 100 000 acres; the problems that confronted Mr. Redman during his fourteen years in charge were varied and difficult. That he was eminently successful, both technically and diplomatically, may be noted from the fact that both his superiors and the project farmers were warm in their praise of his services. His assignment to the San Carlos Project was in direct recognition of his ability to design, construct, and maintain an irrigation project with a minimum of expense. It may be truthfully said that his work was his chief interest in life, and that he gave a full measure of service to his employers.

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[1] Memoir prepared by J. A. Fraps, Assoc. M. Am. Soc. C. E.

While Mr. Redman was connected with the San Carlos Project only a few months more than a year, in that brief period he made many valuable contributions to the ultimate success of the work. He inaugurated many policies and methods which will doubtlessly remain unchallenged throughout the life of the project. Under his guidance, plans were in the preliminary stages for the complete development of the water resources controlled by the San Carlos Project; it is regrettable that their complete details could not have been carried out by the original designer.

It is particularly significant that Mr. Redman was an engineer of broad experience, ever seeking to add to this store. This attitude evidenced itself to his superiors, and, when he requested to be transferred to the San Carlos Project, was instrumental in their granting his request. His work was one of heavy responsibilities—responsibilities that were never shirked or avoided. His associates found keen pleasure in their work with him; his ability to fathom the essential features of a problem and to reach an accurate decision was worthy of imitation.

Perhaps no more difficult problem exists than the maintenance of friendly personal relations between an irrigation project manager and the farmers, but it is significant that after fourteen years on the Wapato Project, some of which were dangerously dry, Mr. Redman counted more close personal friends among the farmers than he did at any time during his stay. No greater tribute can be paid to his integrity, fairness, and diplomacy.

He is survived by his widow, Mrs. Lena Elizabeth (Burchfield) Redman, to whom he was married on June 1, 1910.

He took an active part in Masonic work, and was a Thirty-second Degree Mason. He was also a member of the Rotary International, taking particular interest in its service work.

Mr. Redman was elected a Member of the American Society of Civil Engineers on September 11, 1926.

PHILIP JACOB REICH, M. Am. Soc. C. E.[1]

DIED MAY 31, 1933

Philip Jacob Reich was born at Louisville, Ky., on January 13, 1879, the son of Julius and Louise (Dueser) Reich. He was educated in the public schools of Louisville, completing his studies there by being graduated from the Louisville Manual Training School. His further education was received at Purdue University, at Lafayette, Ind., from which he was graduated as a Bachelor of Science in Civil Engineering in 1899. At that time he was elected a member of Tau Beta Pi, Honorary Fraternity. Julius Reich had a wholesale saddle and harness business, and his son worked with him during his vacations.

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[1] Memoir prepared by F. S. Merrill, M. Am. Soc. C. E.

After his graduation from college, Mr. Reich worked for six months in the Bridge and Building Department of the Illinois Central Railroad Company. In January, 1900, he entered the employ of the Toledo Bridge Company, at Toledo, Ohio, and, during that year, was in charge as Field Engineer of the construction of an entirely new plant for the Company. In 1901, the Toledo Bridge Company was absorbed by, and became a part of, the American Bridge Company, then being organized.

From 1901 to 1906, Mr. Reich was employed as a Squad Foreman at the Toledo Plant of the American Bridge Company, supervising the detailing of all types of steel bridges and structures. He specialized in movable bridges, and became a recognized authority in the designing and detailing of such structures. For a few months in the summer of 1904 this service was interrupted by a period during which he served as Engineer of Design for the Riverside Bridge Company, at Wheeling, W. Va.

In September, 1906, Mr. Reich was made Plant Engineer of the Toledo Plant of the American Bridge Company, and served in that capacity until 1920. During the earlier part of this period a great variety of steel work for Western railroad bridges was going through the plant; in the latter years, there was an increasing percentage of mill and office building work and miscellaneous structures for the ore mines and stone quarries toward the North, to all of which he gave a great amount of personal supervision. He was responsible for the development of a large amount of local business for the plant and was well known and had a place in the industrial and community life of Toledo.

In July, 1920, Mr. Reich was sent to Pittsburgh, Pa., as Division Engineer of the American Bridge Company. As such, he had supervision of the engineering work at the plants at Ambridge, Pa., Canton and Toledo, Ohio, the Shiffler Plant, at Pittsburgh, and of the designing and estimating work at the Pittsburgh Office. As the Ambridge Plant had no equal in capacity, many of the largest and most important bridges in the United States were either all or in part fabricated there, and many important problems in the development of the designs and in the planning of the shop and erection procedure, came to his desk daily for decision. He met and became a friend of a number of the leading engineers of the country, and many came to him for advice on problems connected with various forms of steel construction.

On January 1, 1933, Mr. Reich was appointed Assistant Chief Engineer of the American Bridge Company. With the added responsibilities of that position, and the engineering work at the Pencoyd, Pa., Elmira, N. Y., and Trenton, N. J., Plants (also under his supervision), he continued in the work he had been doing (as Division Engineer) until his death.

Shortly after coming to Pittsburgh, he built a home in the charming suburb of Ben Avon, not far from the Shannopin Country Club, where he spent many of his most pleasant leisure hours, being an enthusiastic golfer.

Although a member of the Lutheran Church himself, he attended services at Epiphany Protestant Episcopal Church, of which his wife and daughter were members. He was a member of the Keystone Athletic Club, of Pittsburgh, the Shannopin Country Club, and the Engineers Society of Western Pennsylvania.

Mr. Reich died suddenly on May 31, 1933, at the Montefiore Hospital, in Pittsburgh, after a severe attack of appendicitis. His passing has saddened the hearts of hosts of his neighbors and associates.

He was married on June 22, 1901, to Florence Ann Whittingham, and is survived by his widow, a daughter, Florence Louise, and a sister, Louise Reich, of Louisville.

Mr. Reich was elected an Associate Member of the American Society of Civil Engineers on June 6, 1907, and a Member on December 5, 1911. He had served as President of the Pittsburgh Section of the Society from May, 1932, until his death.

GEORGE ALFRED JOY RICKER, M. Am. Soc. C. E.[1]

Died November 2, 1933

George Alfred Joy Ricker was born in Portsmouth, N. H., on June 30, 1863. His father, Charles C. Ricker, was a Lieutenant in the United States Navy, having enlisted at the beginning of the Civil War. Lieut. Ricker came of a long line of seafaring ancestors and for two successive generations, his forebears had been prominent, both as owners and operators in the American Merchant Marine. Following the close of the Civil War, Lieut. Ricker was transferred to the China Coast, but soon finding naval life inactive and uninteresting, as compared with commercial shipping, he returned to the United States, resigned his commission, and, with his father, became part owner and Captain of a ship in the general trade. George Alfred Joy Ricker, then six years old, went to sea with the entire family, consisting of Captain Ricker, his wife, a brother, Charles Ricker, then three months old, and himself.

For six years George Ricker's life was spent on shipboard under the direct eye and tutelage of his father. The extent of his travels is of no especial importance. The time in port was relatively so short that little of the advantages of foreign travel pertained to these years. His father's ability as a teacher, however, appears to have been considerable, and his son received the rudiments of a good education, especially in mathematics. A serious handicap existed, however, in the young man's dislike for the sea and his recurrent attacks of severe seasickness with the beginning of each new voyage. His father, who appears ever to have had deeply at heart the human side of family life, decided that his son was not to be a sailor, sold out his holdings, set up an establishment in Liverpool, England, and entered the service of English ship owners.

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[1] Memoir prepared by E. W. James, M. Am. Soc. C. E.

For one year George Ricker had schooling under an unsympathetic and sarcastic English schoolmaster in a private school near Liverpool. His chief memories of this period appeared to hang about the ever-recurring charges of "transatlantic impudence" by a pedagogue who resented the boy's corrections when he referred to "Massagoosetts." Finally, his father was called to take an especially heavy cargo around Cape Horn to California, and, again, it was arranged that the family should accompany him; but he gave George Ricker his choice of going to sea or going to the United States to live with a grandfather in Durham, N. H. The boy chose Durham and returned to America by steamer. There, he entered Durham Academy and had two years more of formal schooling, the last he was to have until he entered the Massachusetts Institute of Technology four years later.

In the middle of his school life at Durham, George Ricker learned by chance from the marine reports in the Boston papers that his father's ship, the *Kate Kellog,* was reported lost with all on board. The boy kept the news from his grandfather, who was then a man of advanced years, but sought out an uncle in Boston, Mass., who at once started inquiries regarding the wreck. It was six months before he learned that he was not an orphan. His father's ship had in fact foundered off the coast of Patagonia, but most of the crew and the Ricker family had managed to reach land. There, friendly Indians assisted them and, for three months, the entire party lived among these Indians, making clothing of skins and endeavoring to make contact again with civilization. After three months the party reached a Spanish penal settlement in Southern Patagonia, and at the end of about six months from the time of the wreck, was picked up by an English ship and taken to Liverpool. Some fragments of the fur garments worn by his mother while in Patagonia were still in the Ricker family at the time of Mr. Ricker's death.

The shock of the experience to his mother was such that Captain Ricker decided to quit a seafaring life and to return to the United States. When George Ricker was fourteen, therefore, he went to Buffalo, N. Y., with his father who was made Superintendent of Elevators for the Erie Railroad Company at that lake port. George Ricker himself went to work for the Company as a Rodman and advanced rapidly, becoming, before he was eighteen, a Division Engineer of that line.

While in school at Liverpool and, later, at Durham, he had disclosed unusual precocity as an artist, and his parents, especially his father, wished him to continue his art studies, but George insisted that he wanted to become an engineer and to go to the Massachusetts Institute of Technology—somewhat reversing the usual sequence. At sixteen years of age, he sought entrance, but was refused because of his age, and he continued with the Erie Railroad Company, studying meanwhile with a "Tech" graduate in Buffalo. As soon as he was eighteen, he entered the Institute and completed the course in three years, but failed to take a degree because he declined to remain at the Institute to complete his thesis. This was because he was offered a place as Chief Topographer on a locating party for the Northern Pacific Railroad Company, then projecting a line from Helena to Butte, Mont.

Mr. Ricker remained with the Northern Pacific Railroad Company only about eight months, because he had become engaged to be married before going West, and refused to take a bride into the life he found in the mining and railroad camps of that time. He appeared to have the same proclivities toward family life that characterized his sea-going father, and, consequently, he returned to Buffalo and opened a private engineering office. From the first he seems to have had unusual success, and a long career in Buffalo was marked by many notable undertakings. He soon became Engineer for the Buffalo Creek Railroad Company and continued to handle its technical work for many years. He built the first concrete grain elevators at that port, located and constructed the famous Gorge Railway at Niagara Falls, and, beginning in the early Nineties, was identified with many electric trolley and interurban projects.

Mr. Ricker promoted the Buffalo Traction Company and was identified with the Buffalo and Depew Railway, a disastrous venture, in which an electric railway company early tried to compete with a powerful steam railroad company. He made the first surveys for the high-tension lines to distribute electric power from Niagara Falls, establishing the Niagara-Syracuse Line. He was also Consulting Engineer on the Twin Lakes Dam, near Leadville, Colo., and spent nearly a year on that project, making important changes in the design of the contracting firm.

From the time Mr. Ricker returned from the West to Buffalo, he had taken a deep interest in civic affairs. For twelve years, he was member of the Civil Service Commission of Buffalo, most of that time holding the position of Chairman, and was long on the Board of Charities of the city. Later, he became active politically in the anti-Tammany faction of the Democratic Party. This appears to have been a natural attitude for an "up-State" Democrat, in New York, thirty-five years ago.

In the election that carried the Hon. Charles Evans Hughes into the gubernatorial chair at Albany, N. Y., Mr. Ricker was offered the place of State Engineer on the Democratic ticket, but because of his sincere opposition to the New York City Democracy, he refused. The entire Democratic ticket except the Governor was elected. This appears to have disgusted Mr. Ricker, but not to have chagrined him especially. He continued his professional work and his civic and political interests and, in time, Mr. Hughes was succeeded by the late William Sulzer as Governor.

In spite of the events that led eventually to Mr. Sulzer's political death, the Governor apparently tried to start his administration right. He determined to take the Highway Department out of politics as far as possible, and selected as his appointee for Chairman of the Commission, John N. Carlyle, the law partner of Robert W. Lansing, later Secretary of State, under President Wilson. Mr. Ricker used to relate how one night he received a long-distance call at his Buffalo home from Mr. Carlyle who offered him the place of First Deputy and conditioned his own acceptance of appointment on Mr. Ricker's acceptance of the place. Mr. Ricker accepted the offer and thereafter for two stormy years had charge of road construction and contracts for the State. He always recounted the events of his life at Albany with much satisfaction. Undoubtedly, the easy money in highway contracts was so seriously curtailed that it peeved the party leaders, and Mr. Ricker always believed that this circumstance had much to do with the political attacks on Mr. Sulzer.

During his two and one-half years as First Deputy, Mr. Ricker introduced the first successful concrete road construction in the State on a large scale and prepared the first set of standard concrete highway specifications used there. He projected and had surveyed the now famous Storm King Highway, designing it as a galleried structure. He did not remain in office long enough to build it, and that inspiring task fell to other hands under changed plans, involving a bench design with stone guard walls.

When Governor Sulzer was successfully impeached, Mr. Carlyle felt that the new Governor should have a free hand, and determining himself to resign advised Mr. Ricker to do the same. In the administration of the Highway Department by Mr. Carlyle, the cost of road maintenance had increased sharply and a turn toward more durable surface types was indicated. This move was becoming popular with the increase in the use of the automobile, and Mr. Ricker, on his resignation, was offered the editorship of *American Motordom*. This work he continued until the adoption of a policy of Federal Aid by the National Government appeared imminent.

This policy was made dependent on the acceptance of the plan by the several States, *ad interim* if necessary by their Governors, and, finally, by the several State Legislatures. It was realized by several of the larger industries at this time, that the automobile and a National policy of grants to the State for road building would greatly expand that activity if continued, and efforts were made to see that the State Legislatures should be brought into line to accept the policy. There was doubt whether certain Middle Western States would accept the Federal Aid, and the Portland Cement Association employed Mr. Ricker to act as a general road promotion man in Nebraska, Kansas, Arkansas, and Missouri. Such activity may or may not have been necessary. Later events have proved the wisdom of the policy of Government aid beyond any question.

When the World War broke out Mr. Ricker was sent to Washington, D. C., as representative of the Portland Cement Association and remained there in charge of the interests of that trade organization for several years, until his transfer to the Association Offices, in Chicago, Ill.

Soon after returning to Washington from Chicago in 1929, when he severed his connections with the Portland Cement Association, Mr. Ricker was made Industrial Consultant of the National Capital Park and Planning Commission. Later, he was appointed Assistant to Dr. John M. Gries, Secretary of the President's Housing Board, and remained in this position for about a year, until the Board was discontinued.

When the Public Works Administration was organized, Mr. Ricker was appointed to the Board of Technical Review, and held this position at the time of his death, on November 2, 1933, following a brief but extremely acute attack of pneumonia.

He was a member of the Western Society of Engineers and of the Technology and Cosmos Clubs, of Washington, D. C.

He was married on November 24, 1887, to Bessie Turner, of Buffalo, and is survived by his widow, and a brother, Charles Ricker, living in Havana, Cuba.

Mr. Ricker was elected a Junior of the American Society of Civil Engineers on April 7, 1886; an Associate Member on May 1, 1895; and a Member on April 3, 1901.

WILLIAM CHARLES SCHNABEL, M. Am. Soc. C. E.[1]

DIED OCTOBER 9, 1933

William Charles Schnabel, the son of August and Elizabeth (Meinschein) Schnabel, was born in Baltimore, Md., on April 18, 1875.

Soon after his graduation from the Grammar School, Mr. Schnabel entered the building construction field, starting at the bottom. Being of a studious mind, he utilized most of his spare time in acquiring knowledge of such sciences as would aid him in his work. He took the Evening Courses in Architecture at the Maryland Institute, from which he was graduated in 1900, having been awarded the First Prize.

From 1900 to 1901 Mr. Schnabel acted as Superintendent of Construction for J. Henry Miller, Builders, and had charge of the erection of the Maryland Penitentiary. He acted in the same capacity for H. B. Henrahan and Son, Stone Contractors; and then became Superintendent for John Waters, Builders.

In 1903 Mr. Schnabel accepted a position as a Structural Draftsman in the employ of Dietrich Brothers, Baltimore. For nine years he was advanced constantly, until in 1912 he was appointed Chief Engineer of this Company, a position he held until the time of his death.

For many years Mr. Schnabel was a Member of the Architectural Commission of Baltimore, and his intimate knowledge of building construction was fully appreciated by his fellow members.

During practically his entire life he was a deep student of all branches of Engineering, keeping himself informed at all times of the latest developments. This hard work and his devotion to his chosen profession, the building trade, and, later in his life, the special branch of Structural Engineering, in which he excelled, brought their just rewards.

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[1] Memoir prepared by Herman F. Doeleman, M. Am. Soc. C. E.

He is survived by his widow, Grace (Stone) Schnabel, of Baltimore, a son, William Thomas Schnabel, of Wilmington, Del., and a daughter, Mrs. Ethel S. Garman, of Philadelphia, Pa.

Mr. Schnabel was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919, and a Member on May 23, 1922.

MORRIS ROBESON SHERRERD, M. Am. Soc. C. E.[1]

Died October 19, 1933

Morris Robeson Sherrerd was born in Scranton, Pa., on December 16, 1865, a son of Samuel and Frances (Hamilton) Sherrerd. As a boy he went to school at Blair Academy, at Blairstown, N. J. He was graduated from Rensselaer Polytechnic Institute, Troy, N. Y., with the degree of Civil Engineer in 1886.

During the years 1887 to 1889, Mr. Sherrerd was engaged in railroad engineering in Pennsylvania, and from 1889 to 1890 he was Engineer in Charge of Construction of the Public Improvement Commission of Troy, N. Y. From 1890 to 1892, inclusive, he was Assistant City Engineer of Peoria, Ill., and, from 1892 to 1895, he served as City Engineer of Troy, N. Y. In 1895, he became Engineer and Superintendent of the Water Department of the City of Newark, N. J., and remained in the positions of Chief Engineer or Consulting Engineer of the City of Newark, until 1926.

Mr. Sherrerd's principal interests and devotion to the time of his death, were connected with the City of Newark, and the public water supplies of the State of New Jersey. He was Chief Engineer of the State Water Supply Commission from 1907 to 1916, during the entire period of its existence, and he was Chief Engineer, or Consulting Engineer, of the North Jersey District Water Supply Commission from 1916 to 1926, during the construction by that Commission of the Wanaque Dam and Reservoir, which was and is the largest single unit of water supply within the State of New Jersey, costing a total of about $26 000 000. From and after the year 1920, he was on the Technical and Advisory Board of the Port of New York Authority and the Russell Sage Foundation Plan for New York City and Its Environs; also, he was Chief Engineer of the New Jersey State Water Policy Commission from its organization until his death which occurred on October 19, 1933.

He had charge, as Engineer, of the complete construction of many small water-works plants, and for a considerable period during his connection with the City of Newark, he acted as the City's Chief Engineer.

Mr. Sherrerd was the author of many engineering reports. He was a member of the American Water Works Association, the New England Water

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[1] Memoir prepared by John H. Cook, M. Am. Soc. C. E.

Works Association, the New Jersey Society for Municipal Improvements, the American Society for Testing Materials, the New Jersey Sanitary Association, the American Association of Engineers, and the Trenton Engineers' Club, of each of which technical societies he was at one time or another an officer or a director. At the time of his death, he was a member of the Board of Trustees of Rensselaer Polytechnic Institute. He was also a member of the Engineers' Club of New York, the Newark Athletic Club, the Essex Club of Newark, the Canoe Brook Country Club, the Forest Hills Field Club, the Downtown Club, the Benevolent and Protective Order of Elks Lodge No. 21, and the City and State Chambers of Commerce of Newark and of New Jersey.

During the greater part of Mr. Sherrerd's professional life his whole interest and ambition were centered in and were a part of the welfare of the City of Newark and the State of New Jersey. He was instrumental in promoting the various statutes of the State of New Jersey which dealt with the construction and operation of public water supplies and the conservation of public water resources, and in respect to legislation governing these matters he was, during his professional career in New Jersey, a greater influence than any other engineer.

Mr. Sherrerd was physically a strong, active man who enjoyed outdoor life; he was a member of many clubs and associations and took great pleasure in playing golf. He was of a active and lively disposition, a man of great force of character, and a friendly and lovable companion.

On July 9, 1912, he was married to Eleanor V. Norris. Mrs. Sherrerd died several years ago. He is survived by an adopted son.

Mr. Sherrerd was elected an Associate Member of the American Society of Civil Engineers on May 3, 1893, and a Member on May 6, 1896. He served as a Director of the Society from 1905 to 1907.

ROBERT HARRIS SIMPSON, M. Am. Soc. C. E.[1]

Died May 18, 1933

Robert Harris Simpson, the son of George and Helen (Gardner) Simpson, was born at Clifford, Pa., on November 8, 1872. His early education was received in the public schools and in Harford Military School, at Harford, Pa. In 1891, he entered Bucknell University, at Lewisburg, Pa., where he completed two years of study, and, after one year out of school, he enrolled as a student at Cornell University, at Ithaca, N. Y., from which he was graduated in 1896 with the degree of Civil Engineer.

In September, 1896, Mr. Simpson entered the employ of the Cleveland, Cincinnati, Chicago, and St. Louis Railway Company and served, successively, as Rodman, Instrumentman, and Inspector, until December, 1898, when he became Resident Engineer in charge of construction of the Union Station at Dayton, Ohio. In January, 1900, he was appointed Division Engineer in general charge of construction and maintenance of track and structures for the Michigan Division, and, later, for the Chicago and Cincinnati-Sandusky Divisions, in the order named. He was Engineer-in-Charge of Surveys preliminary to the reconstruction of the Cairo Division, and after September, 1903, was in general charge of surveys and estimates of cost for grade reduction and changes of alignment.

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[1] Memoir prepared by Orris Bonney, E. G. Bradbury, and F. H. Waring, Members, Am. Soc. C. E.

Although greatly interested in railroad work, in which the future promised much in the way of further advancement, the frequent moving from place to place made a home life almost impossible and caused Mr. Simpson to seek other employment. Accordingly, in June, 1904, he secured a position under the late Julian Griggs, M. Am. Soc. C. E., who was then Chief Engineer of the Engineering Department, Board of Public Service, of the City of Columbus, Ohio. Mr. Simpson's first work was in connection with the design of important sewerage and water-works improvements, including sewage disposal and water purification plants and pumping stations, under the immediate direction of John H. Gregory, M. Am. Soc. C. E. In April, 1906, he was transferred from this work and was made Engineer-in-Charge of the City's interests in connection with the design and construction of grade-crossing elimination projects, involving a total expenditure of approximately $4 000 000. In addition to the construction work during this period, comprehensive studies as to future needs were made and preliminary plans prepared for grade-crossing elimination projects throughout the entire city, many of which have since been built. In January, 1917, he was placed in charge of the preparation of plans and the supervision of construction of the Scioto River channel improvement, designed to prevent a repetition of the disastrous flood of 1913. This project was completed in 1922 at a cost of about $3 500 000.

The faithful and efficient work of Mr. Simpson, under Mr. Griggs and his successor, Mr. Henry Maetzel, was rewarded by his appointment, in September, 1920, as Chief Engineer of the Division of Engineering and Construction, Department of Public Service, of the City of Columbus, in which capacity he was in charge of the design, construction, and maintenance of all paving, sewers, bridges, grade-separation work, and other municipal engineering activities, these duties continuing until his death. Among the works constructed during this period was a comprehensive intercepting sewer project (1928–1933) costing about $3 000 000, in connection with which there were built two storm stand-by tanks, believed to be the first structures of their kind to be placed under construction in the United States, the combined cost of which was about $700 000. A part of the Olentangy-Scioto Intercepting Sewer was designed and constructed to serve as a base for a retaining wall, making possible the construction of a boulevard along the Scioto River adjacent to the Civic Center, and forming a part of the plan for the dedication of the river front to Public Buildings of the State and City. Another section of the same sewer was constructed across the Scioto River in the unique form of a low overflow dam, thus conserving available fall, avoiding the necessity of a depressed section, and maintaining a proper water level in the river. A section of the Alum Creek Intercepting Sewer was built in tunnel, under compressed air, beneath a built-up section of the city, for a distance of about 2.7 miles, the maximum depth being about 70 ft.

Intensive studies also have been in progress since about 1928, under the direction of Mr. Simpson, for a new sewage treatment plant for Columbus, and general plans have been prepared for a plant of the activated sludge type.

In the development of the extensive sewerage and sewage disposal program, Mr. Simpson had the active assistance of Professor John H. Gregory, who has served the City of Columbus as Consulting Engineer on water supply and sewerage for many years.

A work in which Mr. Simpson took a great and particular interest was the municipal airport, known as Port Columbus, the design and construction of which, with the exception of the buildings, were under his direct charge. Built in 1929, at a cost of approximately $800 000, it is one of the most complete and important airports in the United States, and the arrangement of runways, lighting, drainage, etc., has attracted much attention and many visitors. His interest in the Port and in aviation was such that he was an almost daily visitor there to the time of his last illness.

In addition to his duties with the City of Columbus, Mr. Simpson found time to act as Consultant for the City of Newark, Ohio, in connection with grade-crossing problems, this service covering the period from 1911 to 1930. He was also retained, from time to time, by other municipalities as Consultant or Arbitrator. He was the author of numerous papers and articles for engineering and municipal organizations and for technical magazines, principally on the subjects of street construction and airports.

Mr. Simpson was President of the Central Ohio Section of the Society for the term of 1928–1929. He was also a member of the American Society of Municipal Engineers, the Engineers Club of Columbus, the City Officials Division of the American Road Builders Association (of which he was Vice-President for the Central District), the Metropolitan Columbus Commission, and the University Club of Columbus. He was an active member of the Indianola Presbyterian Church which he served as Trustee for several years.

It was generally recognized that no one person had more to do with the material shaping of modern Columbus than Mr. Simpson. He was known as a conscientious and capable public officer, and, following his demise, these facts were set forth in strong terms by action of the City Council, by the local newspapers, and by a memorial broadcast from Station WCAH, Columbus. By those who knew him, Mr. Simpson was most highly esteemed as an upright citizen, a true friend, and a thorough and competent engineer. In the home, he was an ideal husband and father.

He was married, on September 8, 1898, to Cora B. Coleman, of Columbus, and is survived by his widow, their daughter, Mrs. Dorothy (Simpson) Wilson, a brother, Frank M. Simpson, and a sister, Mrs. Jennie (Simpson) Scott.

Mr. Simpson was elected a Member of the American Society of Civil Engineers on April 18, 1927.

RAY HAMILTON SKELTON, M. Am. Soc. C. E.[1]

DIED OCTOBER 29, 1933

Ray Hamilton Skelton was born in Brooklyn, N. Y., on October 18, 1888. His father, Christopher Payson Skelton, was a direct lineal descendant of the Rev. Samuel Skelton who landed at Salem, Mass., in January, 1629. His mother was Frances Alice Kirby.

Mr. Skelton was graduated from the Boys' High School, in Brooklyn, and, later, entered Sheffield Scientific School of Yale University, at New Haven, Conn., where he received the degree of Bachelor of Philosophy in 1909, and the degree of Civil Engineer in 1912. As an undergraduate he was awarded the Tucker Prize for excellence in Sanitary Engineering.

Upon graduation, he took a position as Timekeeper and Cost Analysis Clerk with the Ferro-Concrete Contracting Company; later, he was Assistant Superintendent with the Worcester Salt Company, at Silver Springs, N. Y.

In October, 1911, Mr. Skelton entered the Sheffield Scientific School as a Graduate Student, serving also as Assistant Instructor. During the period, July, 1912, to July, 1913, he was Draftsman with Westinghouse, Church, Kerr and Company, in New York, N. Y. He then entered the employ, as Computer in the Bridge Department, of J. G. White Engineering Corporation, New York City.

From October, 1913, to June, 1917, he was Instructor in Civil Engineering at Yale University. The following three years were spent as Professor of Railroad Engineering at Pei Yang University, Tientsin, China. Returning to the United States in 1920, Professor Skelton accepted an appointment as Instructor in Engineering Mechanics at Yale University.

He spent the summer of 1921 in the United States Bureau of Public Roads, Education Division. From the fall of 1921 until 1925, he was Assistant Professor of Surveying and Engineering at Allegheny College, Meadville, Pa. At this time, he was engaged in Surveying and Bridge Maintenance for Venango County, Pennsylvania. During the summers of 1922 to 1925, he was Instructor in Surveying at the Camp of the Yale School of Forestry at Milford, Pa.

In October, 1925, Professor Skelton accepted an appointment at the University of Maryland, College Park, Md., as Assistant Professor of Civil Engineering, and in October, 1930, he was made Associate Professor of Civil Engineering, a position which he held at the time of his death.

While Professor Skelton was teaching at Allegheny College, in Meadville, he undertook some private work on boundary surveying in the City of Meadville and vicinity. In the execution of this work numerous uncertainties over property lines developed, which aroused his deep interest in this highly specialized branch of surveying. With characteristic thoroughness, he made a determined effort in each case to bring to light, and place in its true position, every fragment of information that he was able to discover, which had any bearing upon the boundary problems on which he was engaged. In attempting to weigh these bits of information, which were of varying importance and often in conflict, he was led to a study of legal principles and established Court procedure bearing upon the settlement of boundary disputes. After several years of intensive application, in which all his spare time was spent in field work and in selecting and classifying a vast amount of legal material, he decided to arrange these data for publication. His book, "The Legal Elements of Boundaries and Adjacent Properties," was published in 1930. The value of this book lies in its careful analysis of numerous Court decisions pertaining to boundaries and the legal rules based thereon. It is a valuable contribution to the subject.

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[1] Memoir prepared by C. S. Farnham and A. N. Johnson, Members, Am. Soc. C. E.

At the time of his death, Professor Skelton was engaged in the preparation of a brief series of lectures on the general subject of the legal elements of boundary surveying, designed primarily to give undergraduate students in civil engineering an appreciation of some of the problems arising in boundary surveying and an approach to some of the basic legal principles involved. One lecture of this series was given at the Fall Session of the Yale Engineering Camp in September, 1933.

Professor Skelton was a member of the Washington, D. C., Society of Engineers, the Washington Yale Club, Yale Engineering Association, the American Association of Engineers, Association of Chinese and American Engineers, Sigma Xi of Maryland, and Tau Beta Pi (Connecticut Alpha); he was also a member of the Old Planters' Society. Since 1927, he had been a Trustee of the University Park, Md., School. He was a member of Christ Church (Protestant Episcopal), at Bay Ridge, N. Y.

Professor Skelton's sincerity and honesty of purpose impressed all who came in contact with him. With his students he entered whole-heartedly in their work, and was a teacher of rare understanding. His was a full and useful life.

In 1918, he was married to Bessie Kirk Wheeler, of Stratford, Conn., who with three children, Ray Hamilton, Jr., Frances Alice, and Eugene Wheeler, survives him.

Professor Skelton was elected a Member of the American Society of Civil Engineers on June 2, 1931.

JOHN RODOLPH SLATTERY, M. Am. Soc. C. E.[1]

DIED SEPTEMBER 23, 1932

John Rodolph Slattery was born at Athens, Ohio, on January 31, 1877. His parents were John Ambrose Slattery, a lawyer with a notable practice in Southern Ohio, and Lena M. (de Steiguer) Slattery. His maternal forebears came of a distinguished family, the de Steiguers, who came to America from Switzerland in 1819 and settled in Southern Ohio. An uncle, Louis de Steiguer, a graduate of the United States Naval Academy, served as an officer in the Navy until his retirement on April 1, 1931. He reached the rank of Admiral and for a time was in command of the battle-fleet. Another uncle, George E. de Steiguer, is a prominent lawyer, residing and practicing his profession in Seattle, Wash.

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[1] Memoir prepared by Robert Ridgway, Past-President, Am. Soc. C. E., and Brig.-Gen. George B. Pillsbury, U. S. A., and Arthur S. Tuttle, Members, Am. Soc. C. E.

Young Slattery received his early education in the Grammar and High Schools of Cincinnati, Ohio. In 1915, the Ohio State University, Columbus, Ohio, in recognition of his achievements, conferred on him the Honorary Degree of Master of Arts. He entered the United States Military Academy, West Point, N. Y., as a Cadet on June 15, 1896, by appointment of Mr. Charles P. Taft, of Cincinnati, a brother of former President William H. Taft, and, four years later, was graduated No. 5 in his Class, an honor indicative of his high standing as a cadet and a student. This high standing gave him the privilege of selecting any branch of the Army in which to serve—he chose the Corps of Engineers. He was appointed additional Second Lieutenant on the day of his graduation, June 13, 1900, and permanent Second Lieutenant, on February 2, 1901, and served continuously thereafter in the Corps of Engineers until he was given a leave of absence to enter the service of the City of New York, on March 31, 1925. Throughout his life he maintained a genuine devotion to, and an appreciation of the debt he owed, his Alma Mater.

On graduating from the Military Academy, he was assigned to duty with the 1st Battalion of Engineers then serving in the Philippines as part of the forces which were establishing order in the Islands and bringing the Philippine Insurrection to an end. With a detachment of Engineer Troops Lieut. Slattery was placed in charge of the rehabilitation and improvement of roads and bridges in Northern Luzon. He was returned to the United States in September, 1902, and took a brief Post-Graduate Course in Military Engineering at the Engineer School of Application at Washington Barracks, Washington, D. C.

In April, 1903, he was made Assistant to Colonel Amos Stickney, Corps of Engineers U. S. A., the District Engineer in New York, N. Y., and until January, 1904, was in charge of the work on the improvement of the tributary channels of New York Harbor. He was then made District Engineer at Honolulu, in charge of river and harbor and of lighthouse construction in the Hawaiian Islands, where he prepared plans for the improvement of Honolulu and Hilo Harbors, and began the work of improving the former. He was promoted to the grade of Captain while on this duty.

In January, 1907, Captain Slattery returned to the United States and was assigned to duty in command of Company L of the 3d Battalion of Engineers in the Second Military Occupation of Cuba, with station at Camp Columbia, near Havana. In June of that year he returned with his Company to station at Fort Leavenworth, Kansas, until March, 1911, when it changed station to San Antonio, Tex.

In July, 1911, he was assigned to duty as District Engineer at Jacksonville, Fla., in charge of the river and harbor improvements in various important harbors on the Florida Peninsula, including Jacksonville, Tampa, and Key West, and a large number of minor port improvements. On February 27, 1913, he received his promotion to the grade of Major. He was transferred in December, 1913, to duty as District Engineer of the Third Mississippi River District, at Vicksburg, Miss., where he had charge of the middle sector of the flood-control works on the Mississippi River, embracing 240 miles on the main stem of the river and, in addition, the works on the Arkansas River. Major Slattery remained on this duty until his services were required in the organization of American forces for the World War in 1917.

In August, 1917, he was commissioned as Colonel of Engineers in the National Army, was placed in command of the 312th Regiment of Engineers, and became Division Engineer of the 87th Division, of which that Regiment formed a part. The Division received its training at Camp Pike, Arkansas, and Camp Dix, New Jersey, and embarked for France on September 5, 1918. Colonel Slattery remained with the Division at Pons, France, until September 30, 1918, when he was assigned on the Staff of Base Sector No. 2, at Bordeaux. On October 19, he was made Chief Engineer of the 7th Army Corps. He was with the Headquarters of the 3d Corps during the final preparations for and the first part of the advance of the First American Army in the final drive of the Argonne-Meuse offensive action. He took part in the preparation for the entry of the 7th Corps into the line, but the Armistice of November 11, 1918, brought operations to an end. After the Armistice, Colonel Slattery was in charge of the engineer operations of the 7th Corps in Belgium and, subsequently, as a part of the Army of Occupation in Germany, with headquarters at Wittlich, until February 6, 1919, when he was taken with a severe attack of pneumonia, brought on by exposure. After a stay in the hospital at Wittlich, he was returned to the United States, arriving on March 25, 1919.

In April, 1919, Colonel Slattery was assigned to duty as District Engineer at Portland, Ore., in charge of the extensive improvements of the Columbia River and its tributary, the Willamette, and the sea-coast fortifications at the mouth of the Columbia. In October, 1919, he was returned to his permanent grade of Major, and received his promotion as Lieutenant Colonel on April 2, 1920. He took the course in the Army School of the Line at Fort Leavenworth, Kansas, from September, 1920, to July, 1921. On graduation from this School he was assigned to duty as District Engineer at Nashville, Tenn., in charge of the improvement of the Tennessee and Cumberland Rivers, remaining on that duty until February, 1923.

Colonel Slattery's last assignment as an officer of the Corps of Engineers was to the important New York District on March 1, 1923, in charge of the improvement of New York Harbor, the Hudson River to Troy, N. Y., East River, Harlem River, and Long Island Sound, and the rivers emptying into it, in the State of New York. He was also a member of the Board of Engineers for Rivers and Harbors and of the New York Harbor Line Board.

On April 1, 1925, Colonel Slattery obtained a leave of absence from the War Department to accept appointment as Deputy Chief Engineer, which had been offered to him by the Board of Transportation of the City of New York, then embarking on a project of rapid transit subway construction estimated to cost, including equipment, about $650 000 000. The project included the building and equipping of 55 miles of railroad route, with about 170 miles of single track, with three 2-tube tunnels under the East River and one 3-tube tunnel under the Harlem River, all to be constructed under compressed air. It comprised the building of a 4-track trunk line from Fulton Street in Lower Manhattan, north under Church Street, Eighth Avenue, and other thoroughfares, to Broadway and 168th Street, a two-track branch continuing to 207th Street and Broadway. Branches lead from the main line into the Boroughs of The Bronx, Queens, and Brooklyn. In addition, a two-track "Crosstown" Line traverses north and south, that district of Brooklyn not now supplied with rapid transit. An extensive system of repair shops, inspection sheds, and storage yards were to be constructed for the maintenance and operation of this project. Colonel Slattery served thenceforth as Deputy Chief Engineer until his death, resigning from the Army on April 15, 1926. The project was in the initial stage of construction when he came to it, and he entered upon his new duties with the accustomed zeal and ability and the dynamic energy for which he was noted, and immediately made his influence felt on the work. At the peak of the work there were about 2 700 engineers, inspectors, and clerical employees in the Engineering Department of the Board.

For the first few years, Colonel Slattery's particular duty was the supervision of the construction in the field, including the dealings with the contractors whose contracts totaled several hundred million dollars. This was a job of no mean proportions and called for the exercise of great mental, nervous, and physical strength, which, fortunately, he possessed in abundance. Those who have not had the experience in similar work can have no conception of the enormous amount of detail involved and, therefore, no true comprehension of the heavy burden carried by the Engineer in Charge. Outside his family, Colonel Slattery felt that his first duty was to his work, and he never spared himself from it. He hardly knew the meaning of a vacation, and the hours he worked each day were long. While work was his recreation, he labored not for its sake, but to accomplish results. A hard worker himself he had little patience with any one who shirked his duties.

When the construction was well advanced toward completion, Colonel Slattery's supervising duties were extended to include the equipment of the line. The Board had decided on the policy that the City should supply the necessary power, the line equipment, and the rolling stock, whether the System was to be operated by the City itself or by a private corporation. This made it necessary to study a number of unusual problems of great importance, among them the question whether the City should build and operate its own power plant or buy power from the Utility Company. It was decided to purchase it, for the first ten years at least, from the New York Edison Company and an equitable contract was executed with that Company. The existing transit companies of New York City were using 25-cycle power. On Colonel Slattery's recommendation it was decided to use 60 cycles, that being in accord with the development of power practice, although many advised against the change. After a few transformer stations had been built, attention was directed to mercury arc rectifiers then in an experimental stage. Following a thorough consideration they were adopted as a substitute for the remaining sub-stations at what promised to be a substantial economy. The type of car to be used was studied in great detail aided by a committee of car designers made up of representatives of the operating companies and of the car manufacturers. There was evolved a type and dimension of car which is giving satisfaction in practice. Mention has been made of only a few of the many intricate equipment problems with which the engineers had to deal, and to all of which Colonel Slattery gave great study and attention.

An attempt was made by the Board to obtain bids for the operation of the System, but when this proved futile and public operation seemed necessary Colonel Slattery was informally designated the General Manager of the System, retaining the position of Deputy Chief Engineer, and proceeded at once to organize the force required for operation. The heads of the various departments were selected by him and were appointed by the Board, politics playing no part in the appointments. Few realize what a task it was to build up an operating and maintenance force complete in all its special departments—having no such organization to start with—for such an exacting and intensive service as a rapid transit subway requires. That an adequate force was built up in a short time that has since proved its efficiency in actual operation reflects great credit on Colonel Slattery. It was probably the outstanding achievement of his career.

The first portion of the "City-Owned Rapid Transit Railroad System" as it is officially known, or "Eighth Avenue Subway," as it is popularly called—the portion in Manhattan extending from Fulton Street north to 207th Street—was opened for revenue operation on September 10, 1932. Colonel Slattery was at the important 42d Street Station when this event occurred, but the nervous and physical strain of the final preparation for it had borne heavily on him and within an hour thereafter he suffered a partial collapse. While he partly recovered from it and returned to work for a few days his physician insisted on his resting at home. Early on the morning of September 23, 1932, he expired suddenly of a heart attack. He gave what he had to the service of his fellow man—no one could have done more.

His untimely death affected his associates very keenly, and the Board of Transportation placed on its records the following resolution:

"*Whereas* in the decease of Col. Slattery, The City of New York and the Board of Transportation of The City of New York are deprived of an exceptionally valuable executive whose engineering and administrative ability was of the highest order and attracted the admiration and respect of all who worked with him and whose loyalty to his associates and his fairness to those with whom he dealt were among his outstanding qualities; and

"*Whereas* the members of the Board and its entire staff feel keenly and deplore the loss of Col. Slattery as a friend and as an associate because of his fine character and high quality of work during his service with them:

> "*Resolved* that the Board of Transportation do and hereby does express its appreciation of the exceptional qualities and accomplishments of Col. John R. Slattery and the manner in which he devoted himself to his duties, and do and hereby does express to Mrs. Slattery and all the other members of Col. Slattery's family its deepest sympathy at their great loss."

Colonel Slattery's recreation was in his family and his work. He considered horseback riding a recreation, but, in his late years, he had no time to indulge in it. When he entered West Point it was his idea to join the Cavalry on graduation, because of his great liking for horses, and it was mostly from a sense of duty that he finally selected service with the Engineers.

In connection with the Annual Meeting of the Society held in January, 1933, a memorial service for Colonel Slattery was held in the beautiful Chapel of the U. S. Military Academy, at West Point, at which several of his associates spoke of his great services to the Nation and its Army, to the City of New York, and to his profession. The service was a simple and impressive tribute to the Soldier-Engineer, who had fought the good fight and left behind him the memory of deeds well done.

A General Officer of the Corps of Engineers, referring to his death, has said:

> "In his long service with the Corps of Engineers he was universally known as an executive of sound judgment and of devoted attention to duty and as an accomplished engineer, He was held in the highest respect and esteem by the entire Engineering Department. His successful accomplishment of the important work with the City of New York was highly gratifying to his brother officers, and his untimely death is a deep loss to his associates in the Corps of Engineers."

A long-time friend writes:

> "Colonel Slattery was a friend of mine almost from the day he reported for duty at West Point in the class below me. From that time, 1896, up until the time of his death, our paths crossed a good many times and each time our friendship deepened. Colonel Slattery was an indefatigable worker, a real leader with unusual ability to organize. To his associates he was always approachable, cordial, open-minded, and co-operative. His practical common sense, judicial poise, and discerning judgment commanded respect and confidence and produced results on every activity that he undertook. He had the courage of his convictions. This he illustrated by his resignation from the assured job and retirement privileges of the Army after twenty-nine years of service therein. The meager pay and changing domicile of Army service offered little opportunity to give his children the education and start in life which he wanted them to have. Consequently, he took the chance incident to a very different kind of work and different associations. The speed with which he won success in his new job demonstrated that he had made no error in judging his own ability.
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> "His greatest interest in life was his family. The welfare and happiness of his family came first in his consideration and lent power to his determination to succeed, a determination which undoubtedly brought him to a premature end from undertaking more than any man could carry. The same sterling qualities that he showed as head of his family entered into his friendships, and made him a friend to be counted upon under all considerations. His death was a great personal loss to me."

Colonel Slattery took an active interest in the affairs of the Society. In 1926–1927, he was President of its New York (now Metropolitan) Section, the strongest Section numerically of the Society. He served as a Director of the Society representing District No. 1 from January, 1929, to the date of his death, and was active in Committee work. Following his death the Board of Direction passed a resolution, which is here quoted in part:

> "It may truly be said that Colonel Slattery gave his life for his profession in his zeal, loyalty, and devotion to his work in the public service. He had a distinguished record as an engineer both in the Army and in civilian life. While taken away in the midst of an active career he leaves behind an impressive record of accomplishments. He was an engineer of high character and integrity and a man of action.
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> "In his death the Society, the Engineering Profession, his many friends, and the community which he served so faithfully, have sustained an irreparable loss."

Colonel Slattery was a member of the Society of American Military Engineers, and was President of its New York Post in 1928–1929; a member of the Municipal Engineers of the City of New York, and its President in 1930; a member of the Terminal Engineers Society, and of the New York County Chapter of the New York State Society of Professional Engineers and Land Surveyors. He was also a member of the Engineers and University Clubs, of New York, and of the Army and Navy Club, of Washington, D. C.

He was married on February 22, 1905, at Visalia, Calif., to Elizabeth Bradley, who survives him, with their three children, Dr. Louis Rodolph Slattery, Dorothea Elizabeth Slattery, and Nathaniel Bradley Slattery.

Colonel Slattery was elected a Member of the American Society of Civil Engineers on October 11, 1920.

DAVID SLOAN, M. Am. Soc. C. E.[1]

DIED JUNE 25, 1933

David Sloan was born at Belfast, Ireland, on July 4, 1847, the son of William and Elizabeth Sloan. His parents came to the United States in 1851, and settled in Ashtabula, Ohio, where he completed his private studies and, afterward, was appointed Town Engineer.

In 1871, Mr. Sloan entered the railway service as Rodman on the construction of the Jamestown and Franklin Branch of the Lake Shore and Michigan Southern Railway (now part of the New York Central System). From 1874 to 1875, he served as Assistant Engineer on the construction of the Eastern Kentucky Railroad, at Grayson, Ky.

In 1876, he returned to the Lake Shore and Michigan Southern Railway Company as Chief of an Engineering Party locating the extension of the

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[1] Memoir prepared by F. L. Thompson, M. Am. Soc. C. E.

Jamestown and Franklin Branch from Oil City, Pa., to Ashtabula, and, in 1877, he was in charge of the development of the harbor at the latter town. From 1882 to 1885, Mr. Sloan served as Assistant Engineer on the location and construction of the Chicago and Atlantic Railway (now part of the Erie Railroad), and after the completion of the line, he was appointed Chief Engineer in Charge of Maintenance. In 1886, he went to the Chicago, Milwaukee and St. Paul Railway Company as Assistant Engineer on the construction of a line from Mitchell to Scotland, S. Dak.

In August, 1886, Mr. Sloan was appointed Assistant Chief Engineer, in charge of the construction of the Chicago, Madison and Northern Railroad (a subsidiary of the Illinois Central Railroad). This road was an important link in the development of the Illinois Central System, connecting Freeport, on the charter line, in Northern Illinois, with Chicago, thus providing for that System direct entrance into Chicago from the west over its own rails. In November, 1888, he was appointed Assistant Chief Engineer of the Illinois Central Railroad System; in August, 1897, Acting Chief Engineer; and, in January, 1898, he was made Chief Engineer.

It was during Mr. Sloan's régime that the campaign for the separation of grade crossings in the City of Chicago became most active, and the Illinois Central was the first railroad company in the city to undertake such work on a large scale. The track-elevation work from Fifty-first Street to Sixty-seventh Street, in preparation for handling the World's Columbian Exposition traffic in 1893; the depression of the tracks along the Lake Front (Grant Park) in 1895; and the elevation of the St. Charles Air Line, with the separation of street and railroad grades between the Central Station and Sixteenth and Clark Streets (including Sixteenth and Clark Streets), in 1897 and 1898, produced some new and complicated situations that were all brought to a successful solution.

This period also marked the beginning of general improvement work over the entire Illinois Central System, such as reduction of grades, construction of additional main tracks, gravity switching yards, and new mechanical and water terminal facilities. All these activities called for the careful selection and employment of a large force of competent engineers, architects, and draftsmen, and the deep personal interest displayed in their welfare by Mr. Sloan, and his Office Assistant, the late William Josiah Karner, Affiliate, Am. Soc. C. E., was rewarded by the loyalty of his large staff no less than by the successful accomplishment of the work.

On April 30, 1903, Mr. Sloan accepted an appointment as Chief Engineer with MacArthur Brothers Company, of New York, N. Y., and Chicago, one of the largest contracting firms in the United States. After ten years of active construction work with this firm Mr. Sloan was relieved of the detail burden and appointed Consulting Engineer for the Company. As a major portion of the Company's activities consisted of railroad construction, in which Mr. Sloan had specialized, his counsel and advice were most valuable. During this period, the firm built railroad lines for many important railroad companies, among them a line for the Chicago, Burlington and Quincy

Railroad Company, through the Wind River Canyon, in Wyoming; for the Rock Island System, in Iowa; for the Virginian Railway Company, including heavy tunnel work, in Virginia and West Virginia; for the Carolina and Clinchfield Railroad Company, in the mountains of Tennessee; for the Northern Pacific Railroad Company, in Montana and Idaho; for the Southwestern Railroad Company, in Arizona; for the Kettle Valley Railway Company, in British Columbia; a line for the Government in Uruguay; and many others.

The entire organization of MacArthur Brothers Company had great regard for Mr. Sloan's ability; a quiet man of the highest character, he did not force his opinions on his coadjutors, and was always wise in his counsel; he was a good business man, and, particularly on railroad work, he had an exceptional knowledge of costs and was a keen competitive bidder.

On December 5, 1873, at Granville, Ohio, Mr. Sloan was married to Sarah F. Parry. They had three children, of whom, two survive him, a daughter, Grace E. (Mrs. T. H. Fullerton), of Wilmette, Ill., and a son, William G., of Princeton, N. J., who served as Chief Engineer for the New Jersey State Highway Commission from 1923 to 1928.

During his latter years, Mr. Sloan divided his time between his daughter and his son; he had a workshop at the home of each, where he produced some fine cabinet work which he presented to his family and friends.

He was a member of the Western Society of Engineers and of the Presbyterian Church.

Mr. Sloan was elected a Member of the American Society of Civil Engineers on January 1, 1896.

ACHESON SMITH, M. Am. Soc. C. E.[1]

DIED JULY 12, 1933

Acheson Smith was born at Port Jervis, N. Y., on November 4, 1878. His father was William Harrison Smith who was Chief Engineer of the Carnegie Steel Company, of Pittsburgh, Pa., until his retirement to private life; his mother was Ellen Brownson (Acheson) Smith. He was educated in Pittsburgh at the public schools, at Park Institute, and at the University of Pittsburgh, and was graduated from the latter in 1900 with the degree of Civil Engineer.

During 1900 and 1901, Mr. Smith was Supervisor, and, later, Assistant Engineer, with the Union Railroad Company of Bessemer, Pa. He then entered the service of the Carnegie Steel Company as Engineer on blast-furnace construction at Isabella Furnaces, remaining at this work until 1903, when he became associated with the United States Steel Corporation as Division Engineer and Superintendent of one of the large bituminous coal mines of the River Coal Company. In this position, which he held until 1905, he was engaged in all the many branches of engineering work connected with the development and operation of a large coal mine.

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[1] Memoir prepared by E. C. Sprague, Vice-Pres., Acheson Graphite Corporation, New York, N. Y.

Acheson Smith was a nephew of the well-known inventor and industrialist, Dr. Edward Goodrich Acheson, and, in 1905, the Acheson Graphite Company which had been formed by Dr. Acheson for the manufacture of graphite electrodes and powders by his electric-furnace process, was just beginning to expand in response to the rapidly increasing demand for these products by the recently developed electro-metallurgical and electrolytic industries. In this year, Mr. Smith joined the Acheson Graphite Company and remained closely associated with, and largely responsible for, its destinies until his death.

On first coming with the Company in 1905 he was engaged on experimental work, but was soon made Vice-President in Charge of Operation and Construction, and, later, Vice-President and General Manager in charge of all activities of the Company. In 1926, he was elected President, which position he held at the time of his death. From the time Mr. Smith was made Vice-President and General Manager, the responsibility for the success of the Company rested almost entirely on his shoulders. It was due to his energy, foresight, and sound business ability that new markets were discovered, increased manufacturing facilities provided as needed, and the Company kept always in an unquestionably sound financial condition.

In 1928, the Acheson Graphite Corporation joined the group of companies forming the Union Carbide and Carbon Corporation, at which time Mr. Smith was elected Vice-President of the National Carbon Company, one of the other associated companies.

For several years preceding his death, Mr. Smith spent considerable time each year in Europe engaged in promoting the affairs of the Acheson Graphite Corporation and, later, of the National Carbon Company. He was well known to many of the prominent industrialists of Europe and in commenting on his passing the *Journal du Four Electrique* states that his death has caused a loss that will be felt in Europe as keenly as it was in the United States.

While Acheson Smith's major interest was the sound development of the Company with which he was so long associated, his broad and keen interest in the general affairs of life led him to become associated with many other activities. He was a Director of the Power City Trust Company, the Carborundum Company, and various other financial and industrial enterprises.

He was associated with many scientific and engineering societies, being Past-President of the Electrochemical Society, and a member of the American Chemical Society, and the American Institute of Mining and Metallurgical Engineers. He was also a member of the American Management Association and of the American Academy of Political and Social Science, and wrote many papers on economic subjects, chiefly on Industrial Management, in which he was considered an authority. In 1926, he received the second prize of $500, offered in 1925 by Mr. Alvan T. Simonds, of the

Simonds Saw and Steel Company, in a contest open to all residents of the United States and Canada, for the best essay on the subject "Your Prosperity and Mine."

Mr. Smith was a member of several clubs, the ones in New York that he most frequented being the Union League and the Chemists.

His interest in good government and his desire to be always useful to the extent of his abilities led him to serve on the first City Council of Niagara Falls, N. Y., under the Commission-Manager form of government in 1915, and, during the World War, on the War Industries Advisory Board. He was active in the Masonic Fraternity and, in 1923, attained the honor of the Thirty-third Degree.

Acheson Smith's attainments in the business, engineering, and social world were indeed noteworthy, but his many friends will long cherish his memory more on account of the broad and sympathetic humanity that pervaded all his thoughts and actions. His sympathy with the man in misfortune and his willingness to help constructively those in need made him hundreds of friends both inside and outside the organizations with which he was connected. He had an unfailing ability to obtain the willing and unswerving loyalty of his associates and to inspire absolute trust in all with whom he came in contact.

Mr. Smith was married on October 12, 1905, to Lucy Wood Carmack, of Brownsville, Pa., who, with one daughter, Betty Brownson Smith, survives him.

From the time of his association with the Acheson Graphite Company in 1905 to 1929, Mr. Smith was a resident of Niagara Falls, N. Y., and took an active part in all movements concerned with the improvement of municipal government and the bettering of living conditions in that city. In 1929, he moved his office to New York, N. Y., and, at the same time, acquired a residence in Greenwich, Conn., which was his home at the time of his death.

Mr. Smith was elected a Member of the American Society of Civil Engineers on January 13, 1919.

RAYMOND HEWITT SMITH, M. Am. Soc. C. E.[1]

Died August 21, 1933

Raymond Hewitt Smith, the son of Frank W. and Anna (Hewitt) Smith, was born in Woodstock, Ohio, on May 9, 1886.

In 1908, on the completion of his preliminary education in the grade and High Schools of Woodstock, Mr. Smith entered the College of Engineering, of the Ohio State University, at Columbus, Ohio. From June to September,

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[1] Memoir prepared by Norman R. Moore, Assoc. M. Am. Soc. C. E.

1910, he was employed with the Engineering Department of the Cincinnati, Hamilton, and Dayton Railway Company. From June, 1911, to June, 1913, he held the position of Assistant on the Engineer Corps on the Columbus Division of the Pittsburgh, Cincinnati, Chicago, and St. Louis Railway (later a part of the Pennsylvania Railroad), continuing in this work on a part-time basis when he returned to college to finish his course of study. He was graduated from Ohio State University, in June, 1914, with the degree of Civil Engineer.

Mr. Smith was described by one of his University professors as a substantial and dependable student who worked steadily at his college tasks. He exhibited these same qualities in his professional work. From June, 1914, to September, 1915, he was Resident Engineer and Inspector on the construction of a sewage disposal plant for the City of Urbana, Ohio. On the completion of this project, he assisted on some water-supply tests for the City of Greenville, Ohio. In October, 1915, he was appointed Resident Engineer and Deputy County Surveyor of Champaign County, Ohio, in which position he was in charge of all State Highway construction and maintenance operations in the County. He held this position until September, 1919, with the exception of three months in 1917, when he served as Captain in the Ohio Signal Corps, at Camp Willis, Columbus, where he acted as Assistant in charge of construction and camp maintenance.

In 1919, Mr. Smith was elected County Surveyor of Champaign County, and served two terms in this office. During this period, he was also State Highway Resident Engineer. At the expiration of his term of office in 1923, he was appointed an Assistant Engineer in the Bureau of Maintenance of the State Department of Highways, at Columbus, serving as Assistant to George F. Schlesinger, M. Am. Soc. C. E., who was then Chief Maintenance Engineer. Later, Mr. Smith was appointed Assistant Chief Engineer of Maintenance in charge of traffic-bound roads, in which capacity he continued until his death.

Mr. Smith's ability first came to the attention of the Ohio State Highway Department through his accomplishments in the development of traffic-bound roads in Champaign County. He had developed this type of road to a point where it had become a vital part of the County highway system. At the time of his appointment to the Central Office, there were many miles of unimproved roads in the State System. Some communities were practically isolated during the winter months. Mr. Smith was faced with the necessity of providing an "all-year" road with the limited funds available. How well he accomplished this task can be attested, not only by the people who were directly benefited, but also by the public in general who travel the highways of the State. Under his direction, approximately 4 200 miles of this type of road were constructed, 2 100 miles of which have been given a bituminous top, 1 300 miles have been oiled, and the remaining 800 miles are still in the development stage.

He was recognized as an authority on this type of construction. He made an outstanding contribution to the welfare of the people of the State, and these roads stand as a monument to his achievement.

The prosecution of his work required Mr. Smith to travel extensively over all parts of the State, and he was well liked and highly respected by the many with whom he came in contact. His advice was sought by many, and he was always ready to help in all activities of the Highway Department.

He was a member of the Masonic Fraternity, a Knight Templar, and a member of the Ohio Engineering Society.

In 1915, he was married to Mary Robison, by whom he is survived. He also leaves a daughter, Janet, his parents, and a sister, Lillian Smith.

Mr. Smith was elected a Member of the American Society of Civil Engineers on December 15, 1924.

SIR FRANCIS JOSEPH EDWARD SPRING, M. Am. Soc. C. E.[1]

Died August 25, 1933

Francis Joseph Edward Spring, son of the Rev. E. Spring, was born at Baltimore, County Cork, Ireland, on January 20, 1849. His early education was received from his father and at Middleton School, County Cork. In 1867, he went to Trinity College, Dublin, for a four-year course leading to the degree of Engineering Licentiate, but in 1870, when still two terms short of the period for the final examination, he learned that the Government of India was offering a number of appointments in its Public Works Service. In the examination for this position he passed second and was among thirteen selected from eighty candidates. He was obliged to abandon the final examinations at his College but, a year later, the University specially conferred on him the degree of Licentiate of Civil Engineering.

Mr. Spring's appointment to the Indian Public Works Department dated from August, 1870, and his first work was in the Province of Punjab as an Assistant Engineer in the Irrigation Branch. After more than two years, he was transferred, at his own request, to the newly constituted State Railway Branch, and, until 1880, had to do with both survey and construction work on various projects in the west of India. After a year's furlough to England, he returned in 1881 and was posted to the Province of Bengal and took part in various surveys connected with the project for what was afterward known as the Bengal-Nagpur Railway, crossing India from east to west. In 1884, he became Under Secretary for Railways to the Government of Bengal. This appointment he held for four years during which time he became a Fellow of Calcutta University, interesting himself closely in technical education and frequently acting as an Examiner in engineering subjects.

In 1888, Mr. Spring became convinced that the secretarial duty of criticizing other men's work was inadequate as professional experience. He obtained a transfer to the Executive Charge of the construction of the railway bridge over the River Chenab in the Punjab. This bridge consisted of seventeen spans of 200-ft steel girders, on piers standing on "wells" or monoliths sunk 77 ft below low water. It carried at rail level a road for ordinary traffic as well as for artillery in case of need. Its cost was about £210 000, of which one-quarter was spent on the training or narrowing of the sandy bed of the river from a width of 2 miles to $\frac{3}{4}$ mile.

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[1] Memoir prepared by Ernest Ifill Shadbolt, Esq., Worcester Park, Surrey, England.

In March, 1890, Mr. Spring was specially selected for transfer to the East Coast Railway in order that he might design and build a bridge over the River Kistna, at Bezwada, India. The work consisted of twelve spans of 307 ft, with piers on stone monoliths sunk to a depth of 88 to 90 ft below low water, the tops of the piers being 43 ft above low water. The first stone was laid in December, 1890, and the first train crossed the bridge in March, 1893. The cost of this bridge, with its river training works, was £235 000. During and after this work, Mr. Spring was in general charge of, at first, 100 miles, and later, 400 miles, of the East Coast Railway which was opened in sections as completed.

In 1893, Mr. Spring was appointed to Bombay, India, as Consulting Engineer for Railways and, in 1894, he was transferred to the similar post of Consulting Engineer for Government Railways in the Province of Assam. Early in 1897 he was again moved, and went to Madras, India, as Consulting Engineer and Secretary to the Railway Department. Here, the outstanding feature of the Government policy was the inauguration of cheap branch railways to facilitate the movement of foodstuffs during famine periods to which, unfortunately, this part of India is subject. During this period he became a Member of the Madras Legislative Council, and also a Fellow of Madras University.

In 1901, Mr. Spring was promoted to the grade of Chief Engineer, First Class. He officiated for a year as Director of Railway Construction, India, and, later, was placed on special duty to prepare a project and estimate for a bridge over the Ganges River, north of Calcutta. This gigantic work was not sanctioned at the time, but was carried out later (on a greatly enlarged scale), by Mr. (later Sir) Robert Richard Gales, M. Am. Soc. C. E., at a cost of £2 342 000. During the last two years of his service, Mr. Spring spent his time on the writing of a book on "River Training and Control" and papers on other technical matters. He also spent a short time on leave in visiting Burma, and also Malaya, arriving back in Madras on his fifty-fifth birthday, January 20, 1904, thereby terminating, under the 55-year rule, his service with the Government of India of just one-third of a century.

A few months before his retirement Mr. Spring received the offer to become Chairman of the Madras Harbor Board which was about to commence certain long-discussed developments. He took up this appointment in February, 1904, and held it till May, 1919, a period of fifteen years after compulsory retirement from Government service under the age limit. He soon induced the Board to consent to his performing also the duty of Engineer, under the title of Engineer-Chairman. During the period of his control, he spent about

£1 125 000 on the improvement and equipment of the Port, converting it from a mere fair-weather refuge to a modern harbor with a depth of 30 ft alongside berths fully equipped and an adjoining area of 14 acres of sheds, warehouses, and railway facilities. The work is fully described in several papers[2] by Mr. Spring read before the Institution of Civil Engineers.

In December, 1911, Mr. Spring was gazetted Knight Commander of the Indian Empire (K.C.I.E.), having been for seventeen years a Companion (C.I.E.). He became a Member of the Institution of Civil Engineers in 1881, and during three years he was a Member of its Council. He was also a member of various other technical or learned societies or institutions in England and in India.

Sir Francis left Madras in 1919 after a total service in India of forty-nine years and three months. During the first five years of his retirement, he was consulted by many widely scattered authorities with regard to proposed harbor or railway works in India, in the Malay States, and in British Guiana, so that he did not entirely withdraw from active work until he reached the age of 75. He was married in 1873 to Charlotte Becher, daughter of S. Townsend, who died in 1930.

After his retirement, Sir Francis went to live at St. Aubin, Jersey, where he took a prominent position in the social life of the place. He equipped an excellent amateur workshop, and devoted most of his leisure to model-making and other work, being President of the local Model Engineering Society and entering a display of his work at a Model Exhibition in London, England.

He retained his active habits for many years, but suffered from an illness in 1932 which much reduced his strength and, after a few months of weakness, he died on August 25, 1933.

Sir Francis was elected a Member of the American Society of Civil Engineers, on January 3, 1894.

ROBERT CHARLES STRACHAN, M. Am. Soc. C. E.[1]

Died April 16, 1933

Robert Charles Strachan was born in Newburgh, N. Y., on October 23, 1862, the youngest of eight children of James Strachan and Susan Isabella (McCullough) Strachan. On both his father's and his mother's sides, his ancestry was pure Scottish—the family name being recorded in the annals of Scotland prior to the Norman Conquest of England.

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[2] "Light Reinforced-Concrete Wharfing Used in the Port of Madras," *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXVI, 1911, p. 427; "The Remodelling and Equipment of Madras Harbour," *Loc. cit.*, Vol. CXC, 1912, p. 89; "Coastal Sand-Travel near Madras Harbour," *Loc. cit.*, Vol. CXCIV, 1914, p. 153; and "Madras Harbour, A Head for the New North Arm Sheltering the Reformed Harbour Entrance," *Selected Eng. Paper No. 39*, Inst. C. E., 1926.

[1] Memoir prepared by Joseph J. Strachan, Assoc. M. Am. Soc. C. E.

Robert Charles Strachan received a common school education at Public School No. 16, in Brooklyn, N. Y. Thereafter, from the age of 14, he was self-supporting; literally "drummed" his way through Mount Pleasant Military Academy, then located in the present Town of Ossining, N. Y.; and was graduated at the age of 20 from New York University, New York, N. Y., in the same class (1883) as his elder brother, the late Joseph Strachan, M. Am. Soc. C. E.

His professional career began even before graduation, assisting the late Charles Benjamin Brush, M. Am. Soc. C. E., on work for the Hackensack (N. J.), Water Company. After leaving New York University, he served the Erie and West Shore Railroad Companies in various capacities from Rodman to Head of Field Party on construction. Later, he was engaged in the design of the superstructure of the Kings County Elevated Railway, in Brooklyn. From 1887 to 1891, he was employed on the design and construction of locks and other work in connection with the widening and deepening of the Eastern Division of the Erie Canal, and on hydrographic surveys of the Hudson River.

Between 1891 and 1894, Mr. Strachan served the Edgemoor Bridge Works, at Wilmington, Del.; Milliken Brothers, at New York, N. Y.; and, as Chief Draftsman, the New York Rapid Transit Commission—thus beginning his long career as a designer of bridges, tall buildings, municipal railroads, and other heavy steel construction.

In his private practice from 1894 to 1897, he examined and reported upon the design of such structures, including the Commercial Cable and the Queens Insurance Buildings, in New York, as well as various railroad and highway bridges. Then, until 1900, he was associated with the late Thomas C. Clarke, Past-President, Am. Soc. C. E., notably in the design of the Willis Avenue Bridge over the Harlem River, in New York, and in the supervision of its construction.

In 1900, the City of New York, through its Department of Bridges, was planning three major crossings over the East River, two to serve Brooklyn and one Queens, in order to give physical effect to the recent political consolidation of those Boroughs with the Greater City. To this end, a notable group of bridge engineers was assembled, including then, and later, R. S. Buck, Oscar Erlandsen, Leon S. Moisseiff, Henry B. Seaman, J. A. Knighton, and the late Austin Lord Bowman, Joseph Oscar Eckersley, H. A. La Chicotte, Kingsley L. Martin, and Robert Charles Strachan, all members of the Society. In that year Mr. Strachan undertook the longest, most arduous, and most important single task of his career—the complete design of the Queensboro Bridge, a structure with two cantilever spans each longer than any of that type in existence at its opening for traffic in 1909, save only the single-track railway bridge over the Firth of Forth in Scotland.

This major bridge program of the City completed, Mr. Strachan continued to take an active part in municipal engineering affairs. After several years as Engineer for the Department of Correction, in charge of construction and maintenance of buildings, he served the Board of Estimate and Apportionment, from 1917 to 1922, as Examining Engineer, reporting upon the feasibility, necessity, and probable cost of proposed projects before that Board.

From 1922 to 1925, he was Designing Engineer with the Board of Transportation on subway and tunnel work. During this period he again undertook the design of a harbor crossing of unprecedented length for its type—the proposed Narrows Tunnel from Bay Ridge to Staten Island, for which alternate designs were prepared for construction by the usual shield method and by the trench method. This project, although practicable from the strictly construction standpoint, was not then financially feasible.

In 1925 and 1926, as Assistant Chief Engineer of the Department of Markets, Mr. Strachan was in charge of the design and construction of all City market buildings in the Borough of Brooklyn. During 1927, he was engaged with the Port of New York Authority, in charge of the preliminary planning of terminal facilities for the George Washington Bridge over the Hudson River from Fort Washington, New York, to Fort Lee, New Jersey.

From 1927 to his death in 1933, he was in private consulting practice with offices in New York, investigating and reporting upon various structural and other projects. He never lost his intense interest in municipal affairs, however, and to the last was active in the Brooklyn Engineers' Club, in the Brooklyn Chamber of Commerce, in city planning, in the proposed revision of the Building Code, and in all matters wherein his wealth of experience and ripened judgment might serve the community which had been practically his life-long home.

Mr. Strachan was the author of many papers and discussions in professional and technical publications, and possessed a flair for unusual applications of higher mathematical analyses to practical everyday engineering problems—as, for example, his use as early as 1912 of nomographic charts to simplify hydraulic and electrical problems and the design of reinforced concrete structures, as well as his later development and publication in 1928 of a "Table of Hyperbolic Radians" to simplify problems involving the catenary.

His was a full life, with manifold interests within and without his profession. He was a lover of music, played the violin and viola, and for some years was a member of the Hoadly Orchestra, a Brooklyn organization of amateur musicians. He was an active Mason, and Past Master of Richmond Hill Lodge No. 892, F. and A. M.; and, at his death, was President of Phi Chapter of Zeta Psi, his college fraternity.

Mr. Strachan, in 1888, married Helen Joslin, of Frankfort, N. Y., who died in 1922. Two of their children (Isabelle, born in 1889, and Wallace Joslin, born in 1901) died in infancy. One son, Joseph Joslin Strachan, Assoc. M. Am. Soc. C. E., of Robin Hill, Tenafly, N. J., survives him, together with three grandsons, Richard, Peter, and Hugh.

Robert Charles Strachan was an able engineer; a man of sterling character, broad culture, and public spirit; a devoted husband and father, and a loyal friend. To have known him was a privilege and an inspiration.

Mr. Strachan was elected a Member of the American Society of Civil Engineers on August 31, 1909.

HOLGER STRUCKMANN, M. Am. Soc. C. E.[1]

Died November 17, 1933

Holger Struckmann was born in Aalborg, Denmark, on April 25, 1878, and spent his boyhood days there.

When he was about eleven years old, his father died, and the family moved to Copenhagen, Denmark, where Mr. Struckmann completed his school education. Thereafter, he entered into an apprenticeship with Nielsen and Winther, and became an expert machinist. He then worked for several years in shipyards in England, returning to Denmark to complete his technical education. He was graduated from the Technical Institute, at Copenhagen, in 1902, as a Mechanical Engineer. Shortly thereafter he emigrated to the United States, arriving in New York, N. Y., on April 25, 1902.

Mr. Struckmann entered the employ of F. L. Smidth and Company as Inspector in the machine shop of the American Clay Working Machine Company, at Willoughby, Ohio, and soon thereafter became Superintendent of the shops. After a brief period in this work, during which he became acquainted with equipment for the cement industry, he accepted a position as Superintendent of the Nazareth Portland Cement Company, at Nazareth, Pa., and, in 1904, a similar position with the St. Louis Portland Cement Company, at St. Louis, Mo. From that time Mr. Struckmann was an important leader in the Portland cement industry. In St. Louis, he became associated with Mr. F. R. Bissell, then President of the St. Louis Portland Cement Company. This was the beginning of a friendship and business association that lasted until Mr. Bissell died in September, 1933. Loyalty to his friends was one of Mr. Struckmann's strongest characteristics, and the death of Mr. Bissell was a severe shock to him.

After a number of years as General Manager of the St. Louis Portland Cement Company, Mr. Struckmann left that Company in 1910 and became associated with the Iola Portland Cement Company, at Iola, Kans., later becoming its President and, at the same time, Vice-President of the Texas Portland Cement Company, which was then closely affiliated with the Iola Portland Cement Company. In all this work, Mr. Struckmann acquired a broad experience both as an Engineer in the construction and operation of cement plants, and as an Executive in handling the difficult problems inherent to the building up of a new industry, such as the Portland cement industry was at that time in the United States.

In 1917, a group of New York financiers organized the International Portland Cement Company, for the building and operation of cement plants in Cuba and the Argentine Republic. Mr. Struckmann was offered and accepted the position as President of this Company and moved to New York City. It was in this capacity that he did his most brilliant work and in which his great experience and knowledge of world-wide conditions were given full opportunity. In a short time this enterprise became so successful that it was deemed advisable to organize a new company, and the International Cement Corporation came into existence in 1919, for the purpose of taking over the three foreign plants in Cuba, the Argentine Republic, and Uruguay, then owned by the International Portland Cement Company, together with the domestic companies of the Texas Portland Cement Company, at Dallas, Tex., and the Knickerbocker Portland Cement Company, at Hudson, N. Y. From this beginning, the International Cement Corporation has grown into its present gigantic size, having in addition to the three foreign plants already mentioned, also established a plant in Brazil. In addition to the four foreign plants it now has eleven domestic plants in the States of Pennsylvania, New York, Indiana, Kansas, Alabama, Texas, Louisiana, and Virginia, with a total annual capacity of more than 20 000 000 bbl of cement.

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[1] Memoir prepared by E. Posselt, Vice-Pres., International Cement Corporation, New York, N. Y.

Under Mr. Struckmann's leadership an organization was built up to manage all these properties under the policies for which he was directly responsible, and no greater monument to his character could be erected than the loyalty, esteem, and affection in which he was always held by the men associated with him in his life work.

Mr. Struckmann died in Copenhagen on November 17, 1933. He had just completed a business trip, visiting the subsidiaries of his Company in the Argentine Republic, Uruguay, and Brazil, and was returning by way of Europe, when he was taken ill while visiting in Denmark.

Mr. Struckmann was a Member of the American Society of Mechanical Engineers, the Larchmont Yacht Club, the Seawanhaka-Corinthian Yacht Club, and the Royal Danish Yacht Club. He had been decorated by the King of Denmark.

His first wife, Ellen Cannon, died in 1921. He later married Carla Clausen, who, with an adopted daughter, Louise, survives him. He is also survived by a sister, Mrs. Inger Knudsen, of Copenhagen, and a brother, Edwin Struckmann, of Kansas City, Mo.

Mr. Struckmann was elected an Affiliate of the American Society of Civil Engineers on November 30, 1909, and a Member on May 7, 1913.

GEORGE FREDERICK SYME, M. Am. Soc. C. E.[1]

DIED NOVEMBER 27, 1933

George Frederick Syme, the son of Andrew and Annie Shepard (Bryan) Syme, was born in Raleigh, N. C., on April 6, 1878. He was the descendant of an old Virginia family which traced its ancestry back to Scotland. His early education was received from private tutors, after which he attended the

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[1] Memoir prepared by a Committee of the North Carolina Section, consisting of John D. Waldrop, B. W. Davis, O. B. Bestor, and C. L. Mann, Members, Am. Soc. C. E.

Raleigh Male Academy. In 1894, he entered the North Carolina Agricultural and Mechanical College (North Carolina State College of Agriculture and Engineering), at Raleigh, and was graduated with the degree of Bachelor of Science in Civil Engineering, in 1898, shortly after his twentieth birthday. Later, he returned to his Alma Mater for post-graduate study, and was awarded the degree of Civil Engineer in 1907.

After his graduation in 1898, Mr. Syme plunged actively into the practice of his profession, and for the succeeding eighteen years, devoted his energies almost entirely to Railroad Engineering. During the remainder of 1898, he was Levelman for the Raleigh and Cape Fear Railroad, now the Fayetteville Branch of the Norfolk Southern Railroad; in 1899, he was Assistant Resident Engineer, on an extension of the Seaboard Air Line Railway, from Norlina, N. C., to Richmond, Va., and Assistant Resident Engineer, with the Peoria and Pekin Terminal Railroad Company, at Peoria, Ill. The "wanderlust" seized him in 1900, when he went to Central America to serve as Instrumentman in Nicaragua, for the Isthmian Canal Commission.

Returning to the United States, Mr. Syme was, in 1901, Topographer on the proposed Virginia Central Railway, on surveys from Elkins, W. Va., to Harrisonburg, Va., and, for a short time, he was engaged by the Louisville and Nashville Railroad Company, for surveys in Christian County, Kentucky. The year, 1902, found him with the Seaboard Air Line Railway Company, first as Resident Engineer in charge of the construction of a bridge, 1 mile long, over the Manatee River, at Manatee, Fla., and then as Locating Engineer from Manatee to Charlotte Harbor, Fla.

For the next two years, 1903 to 1905, Mr. Syme was Resident Engineer for the Coal and Coke Railway Company, first, on location, and, later, in charge of ten miles of heavy construction in the mountains of Braxton County, West Virginia, including one long tunnel, several bridges, and deep cuts and fills, with much rock excavation. Then followed (1905–1906), ten months as Locating Engineer for the Carolina, Clinchfield and Ohio Railway Company, in the Great Smoky Mountains, between Spruce Pine and Marion, N. C., involving the location and design of many tunnels.

Seized again with the "wanderlust," from May, 1906, to May, 1907, Mr. Syme trekked southwestward, and became Locating Engineer for the Santa Cruz Railway, in the Patagonia Mountains, in Arizona, and for the Cananea, Rio Yaqui y Pacifico Railway, in Sonora, Mexico (the Mexican Lines of the Southern Pacific Railway Company). A part of this work, requiring 1% maximum grades in a difficult country, presented a problem that had baffled previous attempts at solution and appeared impossible. His final success attested to his resourcefulness and energy.

Mr. Syme returned to the United States once more, and from June, 1907, to January, 1908, was Division Engineer in charge of the completion of about 110 miles of construction on the Norfolk Southern Railroad, from Raleigh to New Bern, N. C., and, for a short time in 1908, Assistant Engineer with the Florida East Coast Railroad Company on its Miami-Key West Extension. Then followed one and one-half years to July, 1909, as Tunnel Inspector, then

Assistant Engineer, with the Carolina, Clinchfield and Ohio Railway Company, in charge of the design, location, and construction of railroad yards, a switch-back railroad, and other general construction; two and one-half years (1909–1911) as Assistant Engineer, with the Seaboard Air Line Railway Company, on various work between Richmond, Va., and Atlanta, Ga., including office engineering, location, design, and construction of sidings, yards and terminals, buildings, bridges, water stations, coal elevators, etc.; three years (1912–1915) as Locating Engineer, and, later, as Division Engineer with the Norfolk and Southern Railway Company, in charge of the location and construction of about 115 miles of line between Raleigh and Charlotte, N. C., including the reconstruction of some old lines; for a time he also served as Valuation Engineer for the Company during the inventory by the Interstate Commerce Commission.

From August, 1915, to March, 1916, Mr. Syme was Chief of Party, making topographic surveys for the Southern Power Company, for hydro-electric developments at Bridgewater, N. C., and on the Catawba and Linville Rivers and Paddy's Creek.

When the North Carolina State Highway Commission was created in 1916, Mr. Syme was quick to see the great possibilities presented for highway development; and prompted by the desire to serve his own State, and, no doubt, attracted by the opportunity to settle in his native city of Raleigh, he secured a position as Engineer with the new Commission.

Bringing with him a rich and varied fund of experience, he served for more than four years as Constructing Bridge Engineer in charge of building major bridges. During this period he reconstructed the bridge over the Catawba River, which had been partly washed out by the flood of 1916; he built other large bridges; supervised bridge surveys; and engaged in bridge design. These projects required a structural knowledge of reinforced concrete, a subject unknown to the curriculum of his college days. With his wonted thoroughness, Mr. Syme purchased three of the best books he could find on the subject, studied them from cover to cover, and worked out every problem in them. He applied this knowledge to the bridges he was building; discovered cracks occurring in the first spans, which were not accounted for in the textbooks; added reinforcement in subsequent spans; and learned for himself the first principles of longitudinal shear and temperature cracks, and correctly placed reinforcement to meet these conditions.

His fondness for mathematics resulted in a tremendous amount of work done in his spare time. His particular bent was to convert the abstruse formulas of higher mathematics into simpler arithmetical or trigonometric forms, and he exhibited a rare combination of mathematical knowledge and "horse sense" in the production of practical formulas. While he insisted that field engineers should "use their heads," he strove to reduce their calculations to a minimum and to make the methods readily understandable. His investigations required much that he considered drudgery in working out test cases and examples; his accuracy in such work, proved by innumerable checks, was practically perfect.

Mr. Syme remained with the State Highway Commission until his death. He founded the Auditing Department, and employed its first head. In June, 1920, just before the launching of the great program of highway construction in North Carolina, he was made Senior Highway Engineer, with manifold duties. He organized and expanded the Roadway Department, with himself as its head. This Department was charged with the preparation and design of plans and estimates covering 6 000 miles of State highways; in one year, his Department designed and let to contract work aggregating more than $30 000 000 and the total amount let to construction reached the sum of $185 000 000. Mr. Syme prepared the first standard for super-elevating and widening of roadway curves in the State, and supervised the preparation of many standards now used by the Commission. He formulated the organization and secured the establishment of the Locating Department.

He took great pride in his Department, and was ever ready to extol its virtues; sometimes humorously, as indicated by the legend, in his own hand writing, under the glass top of his desk: "The silent force that keeps the balance and preserves the leaven in the Highway Commission, is the Road Department."

In 1928, Governor A. W. McLean appointed Mr. Syme to be State Boundary Commissioner, although he still maintained his duties as Senior Highway Engineer; and in co-operation with the State Boundary Commissioner of South Carolina, he located and monumented the North Carolina-South Carolina State Line from the Lumber River to the sea.

The Society chose Mr. Syme as one of the thirty competent civil engineers in the United States to act as Referee, co-operating with the American Council on Education composed of outstanding educators, to select those American Universities which in their judgment were adequately prepared to give instruction leading to the Doctorate in Civil Engineering.

With unbounded interest in all things pertaining to engineering, Mr. Syme was eminently an organizer. He composed the slogan: "'The three eyes: Organize, Systematize, Economize." He believed in careful advance planning and orderly procedure in the execution of any work, whether a mathematical computation, or a big construction job. A contractor on one of the early important jobs of which Mr. Syme had charge, complained that he did not dare move a wheel-barrow until George Syme had written on it full instructions for its use in red kiel.

Mr. Syme was a strong advocate of professional organizations. From 1907 to the time of entering highway work, he was a member of the American Railway Engineering Association. He was active in the founding of the North Carolina Society of Engineers, and was its President for two terms, in 1930 and 1931; he was a Charter Member, and the first President, of the Raleigh Engineers Club.

Probably no member of the American Society of Civil Engineers took a greater interest in modernizing the art of surveying. In 1932, he was elected a member of the Executive Committee of the Surveying and Mapping Division of the Society. At the Annual Meeting of the Society in New York, N. Y., in 1932, he was appointed Chairman of the following Committees of the Division: Committee to draw resolutions advocating the preparation of topographic maps in connection with all Federal Highway projects in the United States; Committee to prepare resolutions to be presented to Congress dealing with the National topographic mapping situation; Committee advocating the establishment of a Central Mapping Bureau at each State Capitol; and he was re-appointed Chairman of the Committee charged with the preparation of a Manual of Practice in Highway and Bridge Surveying for general use.

He was a member of a Committee on Geodetic Control Surveys; and also a member of a Committee which drew resolutions urging President Hoover to authorize the United States Coast and Geodetic Survey to establish arcs of triangulation throughout the country at 25-mile intervals instead of 50-mile intervals as at present. This idea originated with him; the resolution was presented and approved, and the work authorized.

Mr. Syme also conceived the idea of applying geodetic control to surveys of every description, and attracted nation-wide attention by his paper on "Geodetic Control for North Carolina Highways,"[2] delivered before the Surveying and Mapping Division of the Society at its Annual Meeting on January 21, 1932. This plan, given in considerable detail in his paper consisted: First, in having the Coast and Geodetic Survey monumentation completed in the State; second, in having the State supplement the system with a closer network of monumentation, accurately tied in to the Geodetic System; and, third, the establishment of a central agency, to keep the records of all monumentation and accurate surveys made, and have them available, so that future surveys of every nature may be tied into the Control System. These objectives were attained, and the supplementary monumentation of the State was begun, under the direction of the Department of Permanent Surveys and Records, established in the State Highway and Public Works Commission.

This idea of supplementary surveys under Geodetic Control has been carried into all the States in the country, by and under the direction of the United States Coast and Geodetic Survey, as a Federal project of the Civil Works Administration, and, later, as a State project of the Civil Works Administration. Mr. Syme was appointed State Representative of the United States Coast and Geodetic Survey to direct the work in North Carolina.

Mr. Syme had a colorful, forceful style, and wrote many papers and articles, both for engineering journals and for newspapers. Among his technical publications were the following: "Need for Economic Location of Highways"[3]; "Two Curve Problems Solved"[4]; "Locating Highways in North Carolina"[5]; and, "North Carolina's Geodetic Control System."[6] This last paper was completed just before his death.

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[2] *Civil Engineering,* March, 1932, p. 180.

[3] *Engineering News-Record,* February 19, 1920.

[4] *Loc. cit.,* October 26, 1922.

[5] *Loc. cit.,* December 7, 1922.

[6] *The Military Engineer,* January-February, 1934.

Besides his many professional activities, George Syme was devoted to his Alma Mater, North Carolina State College. He was an active member of the local Alumni Chapter, serving at one time as its President, and always on its principal Committees. In the General Alumni Association he held important committee appointments, and in 1933 completed a three-year term as a member of the Executive Committee; he was for many years, and to the time of his death, Secretary-Treasurer of the Memorial Committee. He was unanimously recommended by the Executive Committee for membership on the Board of Trustees, but as has happened in the case of many other worthy alumni, the Legislature failed to confirm the appointment.

He was a Charter Member of Phi Kappa Phi Honor Society of his college. He was First Captain of the Oak City Guards, at the time it was a unit in the State National Guard.

For many years Mr. Syme served as a Vestryman of Christ Protestant Episcopal Church in his native city; a Trustee of the Raleigh Cemetery Association; and a Master Mason in the William G. Hill Lodge, of Raleigh.

In 1908, he was married to Harriet Haywood, of Raleigh, who survives him. Surviving also are a brother, Andrew Syme, of Raleigh, Traveling Freight Agent for the Seaboard Air Line Railway Company, and a half-sister, Mrs. Blanche Gilliam, of Petersburg, Va.

George Syme was an ardent and indefatigable hunter, and, with his customary sense of organization, planned his winter trips months in advance. Ducks, wild geese, deer, and bear held his interest for many a season, but latterly his entire attention was given to hunting wild turkeys. He became expert with his turkey-call, which he practiced upon assiduously. A lover of hunting dogs, preferably high-bred setters, he owned one or more such companions the greater part of his life. Strong, vigorous, and keen, he taxed the stamina of the best of his fellow huntsmen. It was in the pursuit of his favorite sport that he met his death. In vaulting over a stone fence, topped by wires, he struck one of the hammers of his pet double-barreled gun against a stone; the barrel was discharged, taking effect in his head. Death was almost instantaneous.

It is not easy to give briefly an adequate tribute to this man who combined in one character so many admirable qualities. Free from envy, unassuming, courageous; a loyal friend, and a boon companion; sympathetic, and practically helpful to the many who appealed to him; scholarly, and of unblemished personal integrity; "the elements were so mix'd in him, that Nature might stand up and say to all the world, 'This was a man!'"

The North Carolina Society of Engineers, at its meeting on January 19, 1934, passed the following Resolution, which was unanimously adopted by the Society:

"*That Whereas,* since the last meeting of the North Carolina Society of Engineers we have suffered the loss of Mr. George Frederick Syme, one of our most beloved and outstanding members, who was accidently killed by the discharge of his own gun while hunting on November 27, 1933, thus terminating his earthly career; and

"*Whereas,* he had so ably served the Society as President for two terms, in 1930 and 1931, as well as in other positions at various times, and had always been a leader in promoting the welfare of the Engineer as well as efficiently handling any task that he was called upon to undertake;

"*Now Therefore Be It Resolved:*

"(1) That this Society desires to make permanent record of its appreciation of the very valuable contribution which Mr. Syme made, in promoting and fostering the work of the Society, and in promoting the welfare of the Engineering Profession at large;

"(2) That we deeply feel the loss of one of our most capable and well trained Engineers, an able and conscientious public official, and above all a Southern gentleman, and faithful friend;

"(3) That we deeply sympathize with his bereaved widow in her great loss, and direct that these resolutions be spread upon the minutes of the Society, and a copy thereof furnished Mrs. Syme."

Mr. Syme was elected a Member of the American Society of Civil Engineers on March 12, 1923.

RUSSELL THAYER, M. Am. Soc. C. E.[1]

Died October 21, 1933

Russell Thayer was born in Philadelphia, Pa., on December 24, 1852. He was the son of Judge M. Russell Thayer and Sophia Watmough (Dallas) Thayer. An ancestor on his maternal side was Benjamin Chew, the last Chief Justice of the Province of Pennsylvania. An uncle, George Mifflin Dallas, was Vice-President of the United States under President James K. Polk.

Russell Thayer received his early education at the Episcopal Academy, at Philadelphia, and attended the University of Pennsylvania for one year, 1869-70, leaving that institution to enter the United States Military Academy, at West Point, N. Y., from which he was graduated in 1874. While at West Point, Cadet Thayer served as Color Corporal, First Sergeant, and First Captain of the Corps of Cadets. Upon his graduation, he was appointed Assistant Instructor of Artillery at the Academy, but resigned his commission as an Officer in the Army late in 1874.

In May, 1876, he was commissioned by Governor Hartranft of Pennsylvania, as Brigadier General commanding the Second Brigade of the Pennsylvania National Guard, and during his eight years of service with the National Guard, he was instrumental in re-organizing it completely.

In 1874 Mr. Thayer entered the employ of the Pennsylvania Railroad Company as Civil Engineer, which position he held until June 1875, when he resigned to accept the appointment of Chief Engineer and Superintendent of Fairmount Park, in Philadelphia. He held this position continuously for twenty-three years. During his service in this capacity, he was instrumental in laying out the grounds for the Centennial Exhibition of 1876, and for many of the roads, drives, and other improvements in the Park. Through his efforts many of the old mills were removed from Wissahickon Creek and he extended the boundaries of the park along Wissahickon Creek to the tops of the hills above the drives, thus preserving the wooded hillsides and preventing houses from being erected close to the drives. He also planned the Park Trolley and its bridge at Strawberry Mansion. At the time he resigned this position the Park Commissioners expressed their appreciation of General Thayer's work in a special resolution of thanks.

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[1] Memoir prepared from information on file at the Headquarters of the Society.

In February, 1898, General Thayer resigned his position as Chief Engineer and Superintendent of Fairmount Park to become associated with the United Gas Improvement Company, of Philadelphia, as President of several of its subsidiary companies, which position he held until his retirement in 1916. During his association with this Company he made a number of improvements and inventions to gas-lamps and was instrumental in obtaining a great many contracts for gas lighting.

Ever a man of ideas, General Thayer, in 1885, designed and built a model of a dirigible which was submitted by him to the Ordnance Department of the Army. The plans for this dirigible were approved, and an appropriation of $60 000 for experiments was recommended by Congress. This fund, however, was never allotted owing to the lack of interest in aerial navigation at that time.

After his retirement from public life General Thayer devoted most of his time to his consulting engineering practice, and to a number of inventions which have since become commercially useful, such as a chemical process for the recovery of pulverized free gold that formerly was lost in amalgamation on electrolytic plates, a gyroscope for the control of airships, etc.

He was instrumental in establishing a line of electrically propelled buses on Broad Street, Philadelphia, and, at the time of his death, he was interested in building a tunnel under the Delaware River connecting South Philadelphia with Gloucester, N. J., a high-speed toll road for motors from Philadelphia to Atlantic City, N. J., and the extension of inland waterways by the construction of a canal from Trenton, N. J., to New York Harbor.

General Thayer was a member of the Rittenhouse Club and other social clubs of Philadelphia. He also belonged to Delta Psi Fraternity.

He was married, on April 18, 1876, to Mary Homer Dixon, of Farley, Pa., and is survived by his widow, five sons, Russell, Jr., Eugene, Alexander, Edmund, and Joseph Thayer, and a daughter, Mary Dixon Thayer (Mrs. Maurice Fremont-Smith).

General Thayer was elected a Member of the American Society of Civil Engineers on December 6, 1882.

ORVILLE HICKMAN BROWNING TURNER, M. Am. Soc. C. E.[1]

Died May 16, 1933

Orville Hickman Browning Turner, the son of Evalina and Dr. Luther Turner, was born in Shelby County, Missouri, on November 25, 1862. He received his early education in the rural schools of Missouri, later attending the University of Missouri, at Columbia, Mo., from which he was graduated with the degree of Bachelor of Science, in 1888. He received the degrees of Topographic Engineer, in 1889, Civil Engineer, in 1890, and Master of Science, in 1891.

During the summer of 1889, Mr. Turner began his professional career as First Recorder on Secondary Triangulation with the Missouri River Commission. In June, 1890, he returned to the Commission with the title of United States Assistant Engineer and served continuously in this capacity until 1898. During the early part of this period, he served as First Observer in Charge of Party on Secondary Triangulation in Montana; then from August, 1891, to May, 1897, and during the spring and summer of 1898, he was engaged in similar work along the entire Missouri River from Helena, Mont., to St. Louis, Mo., acting as First Observer in Charge, Precise Levelman, Topographer, and Hydrographer. In August, 1898, Mr. Turner was assigned to the construction of revetment work along the Missouri River, first as Assistant and, subsequently, from August, 1898, to February, 1900, in charge of this work. In June, 1900, he was placed in charge of dike construction on the Gasconade and Missouri Rivers, in which work he continued until the spring of 1901, when he left the service of the Missouri River Commission.

During the year from August, 1901, to August, 1902, Mr. Turner was Resident Engineer on the construction of an extension of the Illinois Southern Railway, including two inclines for transfer of cars across the Mississippi River, near St. Genevieve, Mo., and a 400-ft steel bridge across the Okaw River.

In August, 1902, he became Chief Engineer of the New Mexico and Pacific Railway Company, in charge of the location of 110 miles of railway in the mountains of Northern New Mexico. This work was completed in October, 1902, and Mr. Turner returned to St. Louis, to accept a position as Superintendent of Construction of the Louisiana Purchase Exposition. He remained in this capacity during the Fair Season of 1904 and until January, 1905, at which time most of the temporary Fair Buildings had been removed.

In March, 1905, Mr. Turner returned to New Mexico as Chief Engineer of the St. Louis, Rocky Mountain and Pacific Railway Company (formerly known as the New Mexico and Pacific Railway Company) in charge of the construction of the 110 miles of line previously located. This road opened extensive coal fields in Northern New Mexico owned by the St. Louis, Rocky

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[1] Memoir prepared by L. O. Turner, Deputy Engr., City Engr.'s Office, Los Angeles, Calif.

Mountain, and Pacific Company, by providing direct rail connection between the several mines of the Coal Company and the Atchison, Topeka, and Santa Fé, the Colorado Southern, and the El Paso and Southwestern Railway Systems. On the completion of the actual railway construction, Mr. Turner continued as Chief Engineer in Charge of Maintenance and became Chief Engineer of the St. Louis, Rocky Mountain, and Pacific Company, the parent Coal Company producing from 3 000 to 5 000 tons of bituminous coal daily.

In 1913, the railway was sold to the Atchison, Topeka, and Santa Fé Railway Company, and Mr. Turner became Consulting Engineer for the St. Louis, Rocky Mountain, and Pacific Company, which connection was continued throughout the remaining twenty years of his life.

In 1912, Mr. Turner entered the Consulting Engineering Field in Northern New Mexico, taking charge of the repair and refacing of a 5 000-acre irrigation reservoir near Springer, N. Mex. From 1913 to 1922, and from 1924 to 1927, he was Consulting Engineer for the Maxwell Land Grant Company, a large land grant having many interests in and around Raton, N. Mex.

From 1916 to 1920, Mr. Turner was Supervising Engineer for the Las Vegas Grant Board during the construction of a $500 000 irrigation project, serving 10 000 acres of land near Las Vegas, N. Mex. The project included the construction of an earth dam, 85 ft high and 1 200 ft long, with pertinent reinforced concrete cut-off wall, outlet conduit, and gate tower. The canal system included 12 miles of main canal, largely through solid rock, two inverted concrete siphons, two red-wood flumes totaling 2 400 ft in length, and a concrete-lined water tunnel, 2 600 ft long.

In 1922, Mr. Turner was appointed City Engineer of Raton, and undertook and completed, in the two years following, the paving of about six miles of city streets, costing $500 000.

During the World War, he was appointed State Director of the Naval Consulting Board for Industrial Preparedness of the State of New Mexico, by the Hon. Josephus Daniels, then Secretary of the Navy.

Mr. Turner was connected with many smaller irrigation and engineering projects in Northern New Mexico during the later years of his life, although his gradually failing health served to limit his activities in his professional work.

He was a man of marked personality, keen in his understanding, definite in his views, and firm in his decisions. He took an active interest in civic matters in Raton, where he made his home in 1905. Mr. J. Van Houten, President of the St. Louis, Rocky Mountain, and Pacific Company, with whom Mr. Turner was associated for nearly thirty years, writes of him:

> "He was a man of profound knowledge in his profession, painstaking in everything he undertook; his habits were exemplary and, above all, he had the highest conception of honor and loyalty. His passing was a great loss to the corporation he served so well for many years."

Mr. Turner was a devoted husband and father, generous to a fault. His home was his castle and all his spare moments were spent with his flowers and garden, or with his library.

He was an Honorary Member of the Engineers Club of St. Louis, and a member of the Sigma Alpha Epsilon Fraternity.

He was married on March 11, 1891, to Maude Ionia Hawkins, of St. Louis, who, with two children, Dorothy Turner, now Mrs. J. B. Lampton, of Sapulpa, Okla., and Louis Orville Turner, of Los Angeles, Calif., survives him.

Mr. Turner was elected a Member of the American Society of Civil Engineers on February 2, 1909.

CHARLES OSCAR VANDEVANTER, M. Am. Soc. C. E.[1]

DIED AUGUST 14, 1933

Charles Oscar Vandevanter was born at "Locust Grove," Loudoun County, Virginia, on October 10, 1849, the son of Washington and Cecilia E. (Braden) Vandevanter. He was graduated from the Special School of Engineering at the Virginia Military Institute in the Class of 1869 and was the first student to receive the degree of Civil Engineer from that School.

Mr. Vandevanter's professional career centered about Railway Engineering, and in that field he attained pronounced distinction. His first employment was in 1870 as Rodman on the construction of the Portland and Ogdensburg Railroad. Subsequently, he was Transitman and Engineer in Charge of Surveys and Construction, for the East Broad Top Railway; County Surveyor of Loudoun County, Virginia; in charge of surveys and construction for the Huntington Furnace Railroad; Topographer on the Mexican National Railroad; and Assistant Engineer and Principal Assistant Engineer on the Cripple Creek Extension of the Norfolk and Western Railroad.

He also served as Assistant Engineer with the Washington, Ohio and Western Railroad; Chief Engineer, Shade Gap Railroad; Assistant Engineer, Pennsylvania Company; Contractor's Engineer on the New Croton Aqueduct, New York, N. Y.; Division Engineer, Western Maryland Railroad; and General Manager, Philadelphia and Seashore Railroad, in charge of construction and operation. Mr. Vandevanter was engaged in private practice at Hagerstown, Md.; and was Principal Assistant and Chief Engineer, Western Maryland Railroad, in responsible charge of construction and maintenance of way; Chief Engineer, West Virginia Short Line Railroad; Division Engineer, Blue Ridge Division of the Carolina, Clinchfield and Ohio Railroad; Chief Engineer, United Railways (electric), of Baltimore, Md.; and Assistant City Engineer of Baltimore.

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[1] Memoir prepared by John Carmichael, Esq., Hagerstown, Md.

Mr. Vandevanter retired from active practice in 1915 and returned to his old home at Leesburg, Va., devoting a large part of his time to the study of genealogy and other local history of his native State. He was earnestly interested in these subjects and was a recognized authority in regard to them. The Congressional Library of the United States awarded him a Certificate of Appreciation for the preparation of historical data concerning Colonial homes in Loudoun County.

Of an old Virginia family, Mr. Vandevanter typically exemplified the excellent characteristics of those worthy people. His positive manner was probably a development of his strict Calvinistic creed, for he was a consistent member and an Elder of the Presbyterian Church; but withal he had great charm of manner and was always an interesting companion. It was a pleasure and privilege to serve under him and to be guided by his engineering experience, as well as to claim him as a friend.

His many friends regretted his early retirement, but now realize that he has bequeathed to them the priceless reputation of an engineer, an honest man, and the successful consummation of his undertakings.

Mr. Vandevanter died at Leesburg on August 14, 1933. He was twice married; his first wife was Virginia Kilgour, of Potomac, Md., and his second, Kate C. Elliott, of Newton Hamilton, Pa. Two sons survive him, Braden Vandevanter, an Attorney-at-Law, of Norfolk, Va., and Captain Elliott Vandevanter, Corps of Engineers, U. S. Army.

He was one of the organizers, and became the fourth President, of the Engineers Club of Baltimore, Md. He was also a member of the Holland Society of New York.

Mr. Vandevanter was elected a Member of the American Society of Civil Engineers on September 2, 1896.

HANS AUGUST EVALD CONRAD von SCHON, M. Am. Soc. C. E.[1]

Died December 25, 1931

Hans August Evald Conrad von Schon, the son of Alexander and Louise (Wilde) von Schon, was born on June 25, 1851, at Stolpe, in the Province of Pomerania, Germany.

In accordance with family tradition, the boy was prepared for a military career which began by his attendance at the youthful age of ten at the Prussian Cadet School at Lichterfelde, a suburb of Berlin, Germany. He was graduated from the Royal Prussian Military Academy, at Berlin, in the spring of 1869, and the same year was commissioned an officer in the Prussian and German Army. He served in 1870 and 1871 as 2d Lieutenant, 3d Rhenish Infantry, 28th Regiment, in the Franco-Prussian War. He had a part in the Battle of Gravelotte, and was decorated with the Iron Cross.

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[1] Memoir prepared by Francis C. Shenehon, M. Am. Soc. C. E.

This young Prussian, born close to the Baltic Sea, and near the present Polish Corridor, came to America in October, 1871. It is not improbable that the reason for parting from his comrades of the victorious army that had conquered France, was the tawdry matter of money. Continued life in the German Army required an independent income, as an officer's salary was meager, and his social expenses were great.

The young emigré was courageous, resourceful, and vigorous. He told the writer that in some of these early American days, he made his living shooting ducks. He was naturalized in 1878.

Up to 1888, he was engaged in various engineering works in field and office in several parts of the country. He was also engaged in mining in Utah and California. In August, 1888, Mr. von Schon became Principal Assistant to J. H. Hayden, Chief Engineer of Coal Surveys and Operations in Raleigh County, West Virginia, with headquarters at Raleigh Court House, W. Va.

In April, 1889, he became Principal Assistant to Lancaster Brothers, Engineers and Contractors, of Roanoke and Norfolk, Va., and Birmingham, Ala., and was engaged in plotting land subdivisions, town sites, and designing and constructing iron furnaces, etc. In June, 1890, Mr. von Schon established an engineering office in Fredericksburg, Va., practicing in Virginia, West Virginia, North Carolina, and South Carolina, on surveys, the design and construction of hydraulic developments, electric street railways, and municipal works.

In May, 1893, he was appointed United States Assistant Engineer by the late General O. M. Poe, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., and was detailed on the Lake Survey at Detroit, Mich. In July, 1893, he took charge of a topographical survey or re-survey of St. Marys River, and continued in charge until its completion in October, 1896, at which time he resigned his Federal appointment and became Chief Engineer of the Lake Superior Power Company, Sault Ste. Marie, Mich. It was while the writer and Mr. von Schon were at Sault Ste. Marie—the writer on ship lock construction and river canalization, and Mr. von Schon on topographical work—that they lived at the same hotel, and they and their families were intimate.

The most important engineering work of Mr. von Schon's career was the construction of the great water power development on the American side at the "Soo." At this point St. Marys River, with Lake Superior for its mill-pond, has a normal volume of flow of 70 000 cu ft per sec, with a fall of 20 ft. Practically all this water is used for power purposes, part on the Canadian side and part on the American side. The ship locks require a small quantity of water during the season of navigation, and fish life in the rapids is protected by a small flow. The bulk of the flow turns turbine wheels. Mr. von Schon's administration of the construction of this large American development was Bismarckian, dominant, efficient, and methodical. The late Alfred Noble, Past-President and Hon. M. Am. Soc. C. E., was one of the Consulting Engineers on this project.

After this work was completed, in 1904, Mr. von Schon engaged in consulting practice in Detroit, Mich., specializing in hydraulics. For a number of years, he edited a technical journal, *The Water Power Chronicle,* which title was later changed to *Water Chronicle;* and his book entitled, "Hydro-Electric Practice," was published in 1908. His consulting practice embraced some connection with Muscle Shoals, water-power development on the St. Joseph River, in Indiana, a water supply system for Highland Park, Mich., a suburb of Detroit where he made his home from 1904 to 1916, and other projects.

During the early part of the World War, due to his military training and familiarity with German military methods, he acted as an interpreter of war reports for the *Detroit Journal,* a newspaper in which he had a daily article on the military situation in Europe.

In 1916, Mr. von Schon retired from active business and moved to Northern Virginia, just outside Washington, D. C. He resided there until the end of 1920, when he removed to South Bend, Ind., where he was appointed Water Power Commissioner to settle disputes between the several owners of water-power plants on the St. Joseph River. Late in 1929, he moved with Mrs. von Schon to his daughter's home in Newark, N. J., where he died two years later, in his eighty-first year.

Mr. von Schon was a member of the Western Society of Engineers and the Michigan Engineering Society. He was a member of the Protestant Episcopal Church, and was instrumental in establishing St. Albans Chapel in Highland Park, Mich., serving as Warden and Vestryman for several years.

A soldierly man, firmly planted on his feet, with a voice to be heard above the din of battle, he carried through life a militant bearing and decided views on discipline and method. In his family life and among his friends, Mr. von Schon was the kindliest and most genial of men; and that voice, designed to thunder out commands, boomed infectiously in the enjoyment of a bit of humor. He used the English language effectively in speaking and in writing. The writer never heard him speak German. He was 100% American.

He was married at Lancaster, Pa., on June 22, 1887, to Ida Jennings, of Morrow, Ohio, and is survived by his widow, a daughter, Clara Louise (now Mrs. W. O. Leslie), and a son, F. A. von Schon.

Mr. von Schon was elected a Member of the American Society of Civil Engineers on September 2, 1902.

LELAND ROSS WALKER, M. Am. Soc. C. E.[1]

DIED JUNE 23, 1933

Leland Ross Walker was born in Jefferson City, Mo., on October 6, 1885, the son of Judge Wright Walker, of the Supreme Court of Missouri. His early education was obtained in the public schools of that State, and from 1904 to 1908 as a student at the Missouri School of Mines, at Rolla, Mo., and also at the University of Arizona, at Tucson, Ariz.

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[1] Memoir prepared by S. L. Rothery, M. Am. Soc. C. E.

Mr. Walker's first professional work was with the Union Iron and Foundry Company, at St. Louis, Mo., from 1908 to 1910. He was soon made Chief Draftsman on the design of structural steel for buildings. From May to December, 1910, he served as Structural Engineer to A. Leschen and Sons Rope Company of that city, on the design and construction of towers and terminals for aerial tramways, and from December, 1910, to April, 1911, he was Inspector of car castings, with the Commonwealth Steel Company, of Granite City, Ill.

During 1911 and 1912, Mr. Walker was engaged, as Superintendent of the Sunset Mining and Milling Company, in lead and zinc milling activities in New Mexico. In September of the latter year, he moved to Tucson to accept the position of Designing and Office Engineer in the City Engineer's Department, where the design of subways, reservoirs, streets, sewers, and bridges occupied his energy and ability for five years.

In 1917, Mr. Walker accepted a position as Designer with the Trussed Concrete Steel Company, of Youngstown, Ohio. In this position he designed reinforced concrete buildings at El Paso, Tex., and checked branch office designs at Youngstown. He returned to the warmer climate of Southern California the following year, where he was occupied for three years in the design and details of reinforced concrete and timber structures for the irrigation canals of the Imperial Irrigation System—an extensive international irrigation project on the United States-Mexican Border, with headquarters at Calexico, Calif. From March, 1921, to June, 1922, he was engaged in ranching and in the private practice of engineering on concrete design at Jefferson City, Mo.

From June, 1922, to February, 1923, Mr. Walker was with the Arizona State Highway Department as Bridge Inspector. A year later, in April, 1924, he located permanently in Santa Barbara, Calif., joining the City Engineering Staff as Engineer of Sewers and Drainage; in 1925 he was made Assistant City Engineer, and became a member of the California Sewerage Works Association. He remained in this position until his last illness which culminated in rheumatic fever producing heart failure.

Meticulous and thorough in planning, and design Mr. Walker gained the respect and appreciation of those who worked with him and depended on him for reliable and detailed data so necessary for properly planned construction. Of a quiet and retiring disposition, he made but few close friends to whom his sincerity was always dependable. He was a member of the Sigma Nu Fraternity.

In June, 1915, he was married to Suzanne Lee Feighan, in Los Angeles, Calif., who survives him.

Mr. Walker was elected an Associate Member of the American Society of Civil Engineers on November 9, 1920, and a Member on July 16, 1928.

WILLIAM THOMAS WALKER, M. Am. Soc. C. E.[1]

Died February 25, 1932

William Thomas Walker, the son of George Morton and Zipporah (Maxwell) Walker, was born on June 28, 1874, at Lawrence, Kans. He attended the Lawrence High School and was graduated from the University of Kansas, at Lawrence, in 1898, with the degree of Bachelor of Science in Electrical Engineering.

His entire life was spent in an engineering atmosphere. His father and an uncle were Civil Engineers. After leaving High School, Mr. Walker spent a short time as Draftsman and Rodman for the Jacksonville Southeastern Railway Company, Jacksonville, Ill., before entering the University of Kansas. His summer vacations while in college were spent in engineering work of some nature.

Mr. Walker's entrance into the field of engineering after his graduation was with the Kansas City Belt Railway Company where he held the position of Assistant Engineer of Maintenance of Way from June, 1898, to September, 1899. At this time he returned to the University of Kansas and received his professional degree of Civil Engineer in May, 1900.

Subsequently, he went to Kansas City, Mo., and was with Waddell and Hedrick, Construction Engineers, from May, 1900, to July, 1903. He then served as Designer of Foundations for the South Side Elevated Railway of Chicago, Ill., from July, 1903, to September, 1904; as Construction Superintendent for the George W. Jackson Company, of Chicago, from September, 1904, to January, 1908; as Construction Superintendent for the Falkenau Construction Company, of Chicago, from January, 1908, to March, 1910; and from March, 1910, until his death, as Construction Engineer and General Superintendent of Construction for H. M. Byllesby and Company, of Chicago.

During Mr. Walker's thirty-two years of active engineering practice, his contributions to and accomplishments for engineering were varied. His most important accomplishments were: The foundations and superstructure designed for the new third track and elevated yards, of the Illinois Central Railroad, at 61st Street, Chicago; Designer and Engineer in Charge of a complete Starch Mill for the Corn Products Company at Argo, Ill.; Construction Engineer on a hydro-electric development at Big Fork, Mont.; Construction Engineer for the Northern States Power Company on a hydro-electric development with difficult foundation conditions at Coon Rapids, on the Mississippi River, near Minneapolis, Minn.; the building of the High Bridge Steam-Electric Power Station, at St. Paul, Minn.; a hydro-electric power house and a dam across the Chippewa River at Chippewa Falls, Wis.; a steam-electric power plant at Granite Falls, Minn.; additions to the Riverside Station at Minneapolis, which were considered by him to be the most hazardous and difficult of his undertakings; and the construction of numerous high-tension transmission lines and electric sub-stations.

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[1] Memoir prepared by G. E. Loughland and H. H. Watson, Assoc. Members, Am. Soc. C. E.

Mr. Walker not only contributed his engineering skill in a most generous manner to the Company with which he was connected for a long term of years, but also to various organizations with which he was affiliated, to the city, and to young engineers who came to him for advice. He was a man who was beloved by every one with whom he came in contact and especially by the men under his direct supervision, and his co-workers.

He was a member of the National Electric Light Association, serving as a member of the Accident Prevention Committee, and a member of The Engineers Club, of Minneapolis. He was also a member of the Golden Valley Golf Club, Minneapolis, the Minneapolis Athletic Club, the Minneapolis Kiwanis Club, and the Masonic Fraternity.

Mr. Walker was married on May 7, 1901 to Daisy Drake, who, with a daughter, Jane Fee, survives him. A son, William, Jr., was killed in an aeroplane crash on March 5, 1929.

He was President of the Northwestern Section of the Society in 1922 and, in 1929, a member of the Committee of the Power Division on Ice as Affecting Power Plants, which made a study of action and means of combating frazil ice on intake racks of steam power plants.

Mr. Walker was elected a Member of the American Society of Civil Engineers on June 30, 1911.

JOSEPH HOUSTON WASSON, M. Am. Soc. C. E.[1]

Died January 7, 1934

Joseph Houston Wasson was born in Smithville, Ark., on January 21, 1885, the son of Joseph Houston and Lucy (Lane) Wasson. His father was a prominent citizen of Lawrence County where he was engaged in agriculture throughout his life.

Mr. Wasson was educated in the public schools of Smithville, and received three years of engineering training at the University of Arkansas, at Fayetteville, Ark. In 1912, he was graduated from the University of Wisconsin, at Madison, Wis., with the degree of Bachelor of Science in Civil Engineering.

After his graduation Mr. Wasson was engaged by the Wisconsin Railroad Commission in the investigation of valuation and service. In 1913, he was employed by the Gogebic Water Power Company; in 1914, by the Great Northern Railway Company; and from 1914 to 1917 by the Interstate Commerce Commission in the valuation of railway track structures.

In May, 1917, he enlisted in the United States Army. In September of that year he was commissioned as Captain and in July, 1918, as Major. He served at Fort Benjamin Harrison, Indiana, Fort Leavenworth, Kansas, Camp Taylor, Kentucky, Camp Humphreys, Virginia, and Camp Kearney, California.

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[1] Memoir prepared by George H. Fenkell and W. C. Hirn, Members, Am. Soc. C. E.

After the close of the World War, Major Wasson returned to his work with the Interstate Commerce Commission in which he was engaged from December, 1918, to January, 1920. In March, 1920, he was employed by the Michigan State Highway Department as Assistant District Engineer in charge of Highway Construction.

In May, 1921, he became Field Engineer for the Portland Cement Association in Michigan and was engaged in the promotion of a wider and better use of cement through investigation and educational work.

In 1927, Major Wasson was engaged by the Peerless Portland Cement Corporation as Sales Engineer and was acting in this capacity when he was killed in an automobile accident while driving from Detroit to Lansing, Mich., on January 7, 1934.

After his discharge from military service, Major Wasson was an active member of the Society of American Military Engineers, and was promoted to the rank of Lieutenant Colonel, 310th Engineers Reserve Regiment, in 1923, and to that of Colonel, in 1930.

Colonel Wasson's most outstanding engineering accomplishment resulted from his diligent services in promoting a more careful and intelligent use of Portland cement. His honest and earnest efforts, intelligently applied, resulted in an improved quality of Portland cement concrete and in a wider use of this material where such use was in keeping with good engineering design. He fearlessly denounced the misuse of Portland cement concrete and as fearlessly promoted its use where his judgment dictated it to be proper and for the best interests of all concerned.

His many friends in the Engineering Profession always found him a most capable and willing worker in Society affairs. He was a Past-President of the Detroit Section; and a member of the Detroit Engineering Society, the Michigan Engineering Society, American Concrete Institute, and Society of American Military Engineers. He was also a member of the Masonic Fraternity.

He is survived by his widow, Lucy Lee (Hubbel) Wasson, to whom he was married on June 20, 1918, a son, John Powell, four sisters, and two brothers.

Colonel Wasson was elected an Associate Member of the American Society of Civil Engineers on April 3, 1922, and a Member on January 16, 1928.

ROBERT MALCOLM WATSON, M. Am. Soc. C. E.[1]

DIED MARCH 4, 1934

Robert Malcolm Watson, was born at Jersey City, N. J., on September 21, 1855, the son of Kenneth and Harriet (Douglas) Watson. He was educated in the public schools of Jersey City, and was graduated from Cooper Union, New York, N. Y., with the degree of Civil Engineer.

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[1] Memoir prepared by Harriet E. Watson and Russell S. Wise, Assoc. M. Am. Soc. C. E.

His first employment was in 1875, with the firm of Ruggles and Kimberly, in Hudson City, N. J. Later, he was appointed to the Bureau of Engineers and Surveyors in Jersey City and was engaged in general municipal improvements.

From 1875 to 1880, Mr. Watson served on the staff of the late Walter Katté, M. Am. Soc. C. E., as Assistant Engineer with the New York Elevated Railroad Company, on the location and construction of the Third Avenue and Ninth Avenue Elevated Railroads, in New York City.

Mr. Watson served again under Mr. Katté, from 1880 to 1884, as Assistant Engineer on surveys for, and location of, the New York, West Shore and Buffalo Railroad, between Weehawken, N. J., and Fort Montgomery, N. Y.; as Resident Engineer in charge of construction from Ridgefield Park, N. J., to Haverstraw, N. Y.; and as Engineer in the Maintenance-of-Way Department.

In 1884, he was Engineer in charge of surveys and location for the Brunswick and Western Railroad in Georgia and Florida, and for a part of 1885, he was with the New Jersey State Board of Assessors as Engineer in charge of the valuation of railroads in Northern New Jersey. In 1885 and 1886, Mr. Watson served as Resident Engineer for Brown, Howard, and Company, on the construction of the Croton Aqueduct from Croton Valley to North Tarrytown, N. Y. He then went West and in 1886 and 1887 was employed as Resident Engineer with the St. Louis and San Francisco Railroad Company on construction work in Indian Territory and Northeastern Texas. Returning East, Mr. Watson was engaged in 1887 and 1888 on surveys for and location of the extension of the Buffalo, Rochester, and Pittsburgh Railroad between Bradford and Johnsonburg, Pa.

In 1888, Mr. Watson formed a partnership with the late Colin R. Wise, M. Am. Soc. C. E., which continued until the death of Mr. Wise in 1924. Mr. Watson continued the business under the old firm name until his death. The firm was engaged in general municipal engineering, designing, and construction, and its members served as engineers for several municipalities in New Jersey. Mr. Watson designed and built several sewage disposal plants in Northern New Jersey, and was employed as Resident Engineer on the construction of part of the Passaic Valley Trunk Sewer.

As a member of the firm of Wise and Watson, Mr. Watson occupied a leading position professionally in Northern New Jersey. He served as Borough Engineer of Rutherford, N. J. (where he resided), for more than thirty-five years. He was also Borough Engineer for Carlstadt, East Rutherford, Union Township (now Kingsland and Lyndhurst), and North Arlington, and designed the water and sewerage systems of these places during his administration.

In 1929, Mr. Watson was employed by the New Jersey State Meadow Reclamation Commission, and made a report to that body on river improvements along the Passaic River.

The record of Robert Malcolm Watson is one of honorable professional achievement and faithful civic service. For seven years, he was a member of the Fourth Regiment, New Jersey National Guard. As a Republican, he was active in politics and a loyal supporter of his party.

The professional organizations to which Mr. Watson belonged were the American Society for Municipal Improvements (serving on several Committees of this Society, his latest activity being the Committee on Airports, of which he had charge in Northern New Jersey), the New Jersey Sanitary Association, and the New Jersey State Sewage Association. He was a member of the Odd Fellows, the Benevolent and Protective Order of Elks, and the Rotary Clubs, of Rutherford.

He was married on October 31, 1892, to Charlott M. Woolley, who died in 1912. There were no children.

Mr. Watson was elected a Member of the American Society of Civil Engineers on April 18, 1916.

WILLIAM RICHARDSON WEBSTER, M. Am. Soc. C. E.[1]

Died January 11, 1933

William Richardson Webster was born in Philadelphia, Pa., on October 5, 1855. After an early education received in preparatory schools in that city, he entered, in 1871, the Massachusetts Institute of Technology, at Boston, Mass., from which he was graduated in Civil Engineering in 1875, at the age of 20 years.

During his early years Mr. Webster was employed by bridge building and by iron and steel-making companies. In 1878 and 1879, he was with the Edgemoor Bridge Company, at Edgemoor, Del., which he left in 1880 to join the Phœnix Bridge Company, at Phœnixville, Pa. Later, in 1880 and in 1881, he was employed by the Pittsburgh, Cincinnati, and St. Louis Railroad Company as Inspector of Bridges. From 1882 to 1884 he was Inspector of Materials with Kellogg and Maurice, at Athens, Pa., and was engaged with that firm in general engineering work. Early in 1884, he erected the Aguadores Bridge, for the Jaragua Iron Company, near Santiago, Cuba.

In 1885, Mr. Webster established his own office in Philadelphia for a general inspection service in connection with materials for bridges and other structures. He became, in 1886, Assistant Manager of the Pottsville Iron and Steel Company, at Pottsville, Pa., but, in 1887, returned to private practice in Philadelphia, continuing his work in connection with materials of construction. In 1892, he became Engineer with the Pottstown Iron Company, at Pottstown, Pa., and while with this Company he developed formulas and tables for predicting the physical properties of steel from its chemical composition. For this noteworthy work he received the award of the Scott Medal from the Franklin Institute in 1897.

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[1] Memoir prepared by C. Laurence Warwick, Assoc. M. Am. Soc. C. E., and Messrs. William R. Kay and A. A. Stevenson, Philadelphia, Pa.

From 1895 to 1920 Mr. Webster continued as an Inspecting and Consulting Engineer with his offices in Philadelphia. Because of the reputation in this work that he had acquired with foreign engineers he was engaged upon the inspection of railway materials which went to all parts of the world, notably to the Argentine, Brazil, Chile, Cuba, and Japan. Among the important structures with which he was connected in the United States were the Cincinnati and Covington Bridge, over the Ohio River, the Blackwells Island Bridge, over the East River, New York City, and the steelwork of the Pennsylvania Railroad Station and Tunnels, in New York. During these years, Mr. Webster also took an active interest in other lines of engineering work. He had a prominent part in the work that led to the grading and sale of pig iron by its chemical analysis instead of its fracture. He also invented a machine for heading eye-bars in one operation, which was a great improvement over former methods of manufacture and gave a more dependable product.

During the World War, Mr. Webster made an important contribution to the production of ships. While, in Washington, D. C., in 1917, he was informed by a Naval official that the country's production of wrought-iron anchor chains was very much below war needs. Mr. Webster suggested the use of high-grade, cast steel produced in electric furnaces, and through his efforts the interest of the National Malleable Castings Company was enlisted, and steel anchor chains was produced with a proof-test higher than ever before obtained.

Mr. Webster was also active in the work of Engineering Societies, particularly the American Society for Testing Materials of which he was a Charter Member. His connection with it began with the year of its organization as an American Section of the International Association in 1898, and he was one of the group of seven to whom was granted the charter incorporating the Society in 1902. He was a member of the Executive Committee of the Society from 1907 to 1912. He was the first Chairman of the present Committee A-1 on Steel, which has developed practically all the Society's specifications for ferrous metals, and served as Chairman until 1913. A great deal of the success of the work of that Committee is rightly attributed to the foundation laid by Mr. Webster and his fellow officers. In 1927, at the exercises in celebration of the Twenty-Fifth Anniversary of the Society's founding, he was made an Honorary Member.

Mr. Webster was also a member of the American Institute of Mining and Metallurgical Engineers which he joined in 1892, a member of the British Iron and Steel Institute, and other technical societies.

He is survived by his widow, Mrs. Elizabeth Bishop (Perkins) Webster, to whom he was married in Athens, Pa., in 1886.

Mr. Webster was elected a Member of the American Society of Civil Engineers on April 5, 1899.

CHARLES HUNTER WEST, M. Am. Soc. C. E.[1]

DIED JUNE 7, 1933

Charles Hunter West, the son of Charles W. West and Kate (Hunter) West, was born on November 25, 1858, at the plantation home of his grandfather, Charles West, in Jefferson County, Mississippi. He was a great-grandson of Cato West, an officer in the Colonial Army during the American Revolution, from Fairfax County, Virginia. Cato West was of English and Scotch ancestry, and settled in Cole's Creek, about 25 miles northeast of Natchez, Miss., on a grant of land from the Spanish authorities in what, later, became Jefferson County, Mississippi. He served as Acting Governor of Mississippi Territory from 1803 to 1805.

Mr. West's mother and father both died before he was five years old, and his early education was limited by the meager facilities of the times immediately after the Civil War. For two years he attended Jefferson Military College, near Natchez, at that time the principal institution of learning in Mississippi. He then went to Greenville, Miss., where he served as a clerk in a store for a short period. During this time he developed a strong desire to become a Civil Engineer and, under the guidance of Major O'Hea, a retired engineer, he began his studies at night. Later, he attended the University of Illinois, and after some interruption was given the degree of Bachelor of Science in Civil Engineering, Class of 1884.

Returning to Greenville, Mr. West began work with a contractor who supplied willows for revetment work on the Mississippi River. Shortly afterward he secured a position with the Mississippi River Commission as Instrumentman and Inspector on levee work, where he remained until 1886. From then until 1892, he was in private practice, engaged mainly in surveying, drainage, and bridge work. In 1892, he returned to the Government service on levee construction and remained there until August, 1898, when he was elected Chief Engineer of the Mississippi Levee Board at Greenville. While in this position he was appointed a Member of the Mississippi River Commission by President Taft, and took his place on this Board on February 5, 1910. As a member of the Commission Mr. West served on many of its important Committees and since 1924 had acted as Chairman of many of them.

His intimate knowledge of the Mississippi River and its problems through a great number of years made his advice sought and his counsel respected. His frankness and fearlessness in expressing his opinions earned for him the respect of his associates. His genial and kindly disposition coupled with his gracious manner endeared him to all who were fortunate enough to know him. Of a naturally retiring disposition, he was never one to force himself into the limelight, yet when the time came to speak, Mr. West was always ready with sound logic to strip all superficialities from the matter under discussion. As typical of Mr. West's approach to any problem, the following lines were written by him, in speaking of his connection with the river work:

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[1] Memoir prepared by W. E. Elam, M. Am. Soc. C. E.

> "It has ever been an interesting and absorbing work, and now, after forty years of close observation and study, I do not consider that I have, by any means, mastered the subject. There are many things that remain to be learned about the river, particularly the lower river from Cairo to the Gulf of Mexico."

Mr. West served as Consultant on many projects, mainly drainage, bridge, and river problems.

He was on the *Norman,* when it capsized on the Mississippi River, about 25 miles below Memphis, Tenn., on May 8, 1925, when an excursion was being run for the Mid-South Section of the Society. His escape was almost a miracle, and it was several months before he fully recovered from the exposure and shock.

The friends of Mr. West often referred to him as "Captain" West. At the University of Illinois, he took the course in Military Science and served as First Lieutenant of the Student Corps. After returning to Mississippi, he entered the National Guard and served as Captain of his Company.

Mr. West was a Mason, a member of the Blue Lodge, Chapter, Council, Commandery, and Shrine. He served as Master of the Lodge and Commander of his Commandery. His connection with the Masonic Bodies was one of his pleasures, particularly in the Commandery, and his services were frequently asked and freely given as long as he was able to attend the meetings. It can be truly said that he lived his Masonry without the hope of fee or reward.

He was a Life Member of the Alumni Association of the University of Illinois; a member of the National Geographic Society; a Life Member of the International Navigation Congresses; a Life Member of the Greenville Chamber of Commerce; an Honorary Member of the Rotary Club of Greenville; and a Charter Member of the Society of American Military Engineers. Mr. West was a member of the Presbyterian Church, but numbered among his friends and associates members of all faiths and religions with never a thought, word, or act of criticism to any other belief. In some of his papers he gives a quotation from Senator Lamar, "Know each other better and we will love each other more," and states that his experience had led him to believe that this doctrine is true as regards the relation between the members of different churches as well as between people in other affairs of life.

Mr. West was married, on January 20, 1886, to Mary Dulaney, of Madison County, Mississippi, who died in 1899. The five children of this union all died before their mother, most of them as the result of an epidemic of scarlet fever in Southeastern Arkansas where Mr. West was employed. In 1908, Mr. West married Miss Birdie Hanway Robertshaw, who survives him. He was devoted to his wife who was his constant companion, and accompanied him on many of his business trips, as well as on pleasure trips.

He was stricken while on the high-water inspection trip of the Mississippi River Commission, and was forced to leave the boat at Helena, Ark. He was brought to Greenville for an operation from which he never fully recovered, passing away on June 7, 1933.

Mr. West was elected a Member of the American Society of Civil Engineers on October 1, 1902, and became a Life Member on November 25, 1928.

HARRY ROBERTS WHEELER, M. Am. Soc. C. E.[1]

DIED OCTOBER 29, 1933

Harry Roberts Wheeler was born at Orange, N. J., on May 16, 1861, the son of George Horace and Cornelia Augusta (Fouquet) Wheeler. He was a graduate of the East Orange, N. J., High School, and thereafter was privately tutored.

Mr. Wheeler joined the Engineering Department of the West Shore Railroad Company while it was being organized in the early Eighties, and was assigned to work on its Hudson River Division.

In 1884, he was appointed Rodman by the Aqueduct Commissioners of the City of New York, and early the following year was promoted to the rank of Leveler. In 1886, he was made Assistant Engineer and served in this capacity until 1889, when he obtained a leave of absence. He then became associated with the Ontario and Western Railroad Company, to take charge of the construction of the so-called "Zig Zag" Tunnel near Walton, N. Y.

From 1904, until his retirement from business in 1926, Mr. Wheeler was connected with the firm of Henry Steers, Incorporated, Contracting Engineers, New York, N. Y., in which organization he served in the capacity of Secretary and Chief Engineer. In occupying this position he had an outlet for his ability. Many of the harbor improvements in New York and vicinity were carried on under his supervision and guidance. His previous railroad experience stood him in good stead, for much of his work consisted of construction in that line. It was chiefly in these fields that he built up his host of friends and acquaintances.

Some of the projects with which Mr. Wheeler was connected, were: The construction of Manhattan Transfer for the Pennsylvania Railroad Company; change of grade and track elevation on the Newark and New York Branch of the Central Railroad Company of New Jersey, in Newark, N. J.; the construction of bulkhead and reinforced concrete warehouses for the Lehigh Valley Railroad Company at Claremont Terminal, Jersey City, N. J.; building of foundations for the Western Electric Company's Kearney, N. J., Plant; the new bridge of the Central Railroad Company of New Jersey across Newark Bay; piers for the Central Railroad Company of New Jersey at Liberty Street, New York City; the construction of the Stamford Dam for the Water Company at Stamford, Conn.; and, during the World War, the construction of piers at the Army Base, South Brooklyn, N. Y., and the building of Pelham Bay Camp.

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[1] Memoir prepared by J. H. Berry, Esq., New York, N. Y.

Mr. Wheeler also served as First Vice-President of the Bayonne Bolt and Nut Corporation, of Bayonne, N. J., and as Secretary of the Eastchester Shore Front Realty Company, of New York.

He was a member of the Society of Naval Architects and Marine Engineers; Sons of the Revolution; the Masonic Order; United States Navy League; National Security League, of New York; Academy of Sciences, of Brooklyn, N. Y.; Institute of Arts; Engineers Club, of New York; Engineers Club, of Brooklyn; Crescent Athletic Club, of Brooklyn; Whitehall Club, of New York; and the New York Machinery Club.

On October 9, 1887, Mr. Wheeler was married to Elizabeth A. Moore, of Poughkeepsie, N. Y. There was one son, Tolman Douglas, who lost his life in France during the World War. The loss of his wife in the summer of 1918, and the death of his son one month later, were blows from which he never fully recovered and which hastened his retirement from business life.

After his retirement, Mr. Wheeler settled at Northport, N. Y., where he resided until his death. Here, the closing years of his life were spent quietly. He found time to pursue the pleasures that his previously busy life would not permit. Boating and fishing were favorite pastimes, and he was also fond of reading. He delighted in visits from his old friends, such occasions always being a source of great enjoyment to him.

He is survived by two sisters, Mrs. Mary L. White, of North Tarrytown, N. Y., and Mrs. F. H. Severance, of Chazy, N. Y.

Mr. Wheeler was laid to rest in the family plot in Rural Cemetery, at Poughkeepsie, alongside his wife and his son. Those who had the good fortune of knowing Mr. Wheeler, are conscious of the loss of both a staunch friend and a capable engineer.

Mr. Wheeler was elected a Junior of the American Society of Civil Engineers on April 4, 1888; an Associate Member on May 4, 1892; and a Member on March 1, 1910.

FREDERICK WILCOCK, M. Am. Soc. C. E.[1]

Died June 15, 1933

Frederick Wilcock, the son of Richard and Lavina Margery (Hodgkinson) Wilcock, was born at Wigan, Lancashire, England, on September 27, 1873. In 1882, when he was nine years old, he came with his parents to New York, N. Y. The family settled in Brooklyn, N. Y., where the boy received his primary education in the public schools. Later, he went to Harvard University, whence he was graduated, in 1900, with the degree of Bachelor of

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[1] Memoir prepared by Harold E. W. Young, Esq., Brooklyn, N. Y.

Science in Engineering. In 1910, he received the degree of Bachelor of Law, and the following year the degree of Doctor of Jurisprudence from the Brooklyn Law School, St. Lawrence University.

After leaving Harvard, Mr. Wilcock took service, in 1901, with the Rapid Transit Construction Commission of New York City, then engaged in building the original Fourth Avenue-Broadway Subway. He retained this connection until his death, through the various mutations of that body which, in its present form, is the Board of Transportation of the City of New York.

His first duties as an Assistant Engineer consisted of design and general work in the office of the Chief Engineer. From 1902 to 1905, Mr. Wilcock was engaged in field work and the preparation of payment estimates in the 42d Street Sector. In April, 1905, his work involved construction records and payment estimates in connection with the Battery Subway and the East River Tunnels. From February, 1906, to September, 1911, he was Office Engineer under the Division Engineer, Fifth Division, on tunnel and subway construction, under the East River and in Brooklyn. His work comprised drawings, payment estimates, emergency field work in compressed air tunnel, field supervision of test borings in the East River for five routes, and preliminary surveys of proposed routes in Brooklyn. Then, for three months, he was employed on emergency work as Field Supervisor of Construction at the commencement of work on the Fourth Avenue Subway, in Brooklyn.

From October to December, 1911, Mr. Wilcock was engaged in field supervision of land test borings for proposed routes in Brooklyn (Seventh Division), and from January, 1912, to June, 1913, in the preparation of contract progress and cost data in the Chief Engineer's Office. In July of the latter year, he became Assistant Division Engineer and from then until March, 1917, was employed by the Chief Engineer in the investigation and compilation of capital expenditures both by the City and the Contractors, for the purpose of determining subway construction costs under the rapid transit contracts for construction and operation.

In April, 1917, Mr. Wilcock became Auditor of Rapid Transit Costs, and, from 1919 to 1921, he was Auditor of the City's capital expenditures under the rapid transit contracts and had charge of various investigations under the Rapid Transit Law.

From 1921, until the time of his death, he was Assistant Engineer in Charge of the Division of Determination of Costs of Subway Construction, in the Chief Engineer's Office, his work in this position involving, among other matters, the investigation of operations under the rapid transit contracts.

It was in 1908 and 1909 that Mr. Wilcock, recognizing that his work increasingly tended to be involved with the various elements entering into the determination of subway construction costs, took up the studies of Law and Accounting in order the better to equip himself to deal with the subject in all its aspects. Because of these studies he was prepared when he came to head the Division of Determination of Subway Costs—involving as it did the passing of contractors' estimates for payment on subway contracts—to conserve the City's interests to the fullest extent.

Mr. Wilcock was a painstaking student of rapid transit construction and operating contracts, and many problems on economic and financial matters were referred to him. Thus, in 1932, when the City failed to receive satisfactory proposals for private operation of the new Eighth Avenue Subway System, and it was decided that the City should undertake its operation, Chairman Delaney, recognizing Mr. Wilcock's varied and extensive knowledge of the factors to be considered in the formulation of a satisfactory plan for municipal operation, appointed him a member of a committee to study and report upon the subject.

That Mr. Wilcock brought unflagging devotion, intelligence, and zeal to his work, and that his efforts received the approbation they merited, is attested by the following resolutions of the Board of Transportation on the occasion of his death:

"*Resolved* That the Board of Transportation of the City of New York hereby records its profound sorrow at the death of Assistant Division Engineer Frederick Wilcock, who died on June 15, 1933, and deplores the loss to the City of one of the ablest and most devoted engineers on the staff of this Board, not only because of his unusual professional ability, but also because of the esteem and affection which all of his associates had acquired for him during thirty-two years of loyal and faithful service for the City with this Board and its predecessors in authority; and be it

"*Further Resolved* That the Board of Transportation hereby expresses its appreciation of the exceptional qualifications of Frederick Wilcock, as Assistant Division Engineer of this Board, and hereby expresses to the family of Mr. Wilcock its deepest sympathy in the loss of a loving husband, father and brother."

In 1911, Mr. Wilcock was married to Jennie Marie Young, a graduate of Barnard College, and the daughter of the Rev. Samuel W. Young, of Port Chester, N. Y. Their union was blessed by four children, three of whom survive: Donald, Ruth, and Richard.

Mr. Wilcock was a member of the American Association of Engineers, the Harvard Engineering Society, the Municipal Engineers of New York, the Harvard Club, and a Charter Member of the Brooklyn Society of Civil Engineers. He was also a Mason, and, for many years, served as a Vestryman and Treasurer of St. Andrew's Protestant Episcopal Church in Brooklyn. At the time of his death he was a member of St. Philip's Protestant Episcopal Church, Dyker Heights, Brooklyn.

Although Mr. Wilcock's zeal led him to pursue the study of law, especially in its bearing in relation to contracts and other aspects of subway construction work, and, in addition, to qualify as a Public Accountant, he yet remained true to his first love for engineering. His interest in the extensive and intricate subway construction work in New York City was untiring, and his experience covered practically the whole field of the construction of the various routes of the three major systems. While his later work confined him to his office, he yet found time to visit and inspect all phases of the construction work of the new Eighth Avenue Municipal System. Although the physical construction of the subways always remained his major interest, the financing and operation of them were matters of the most lively concern to him, and he kept himself at all times fully informed on these important aspects of his work.

A man of quiet and undemonstrative nature, he none the less possessed those qualities of loyalty, honesty, generosity, and modesty which endeared him to all with whom he came in contact, whether as superiors or subordinates. The mingling of these virtues in one person being as uncommon now as ever, it is with profound regret that his passing is recorded.

Mr. Wilcock was elected a Junior of the American Society of Civil Engineers on January 7, 1902; an Affiliate on October 6, 1903; an Associate Member on January 4, 1910; and a Member on June 16, 1924.

HENRY HARRISON WILSON, M. Am. Soc. C. E.[1]

Died May 31, 1933

Henry Harrison Wilson, the son of Henry Joseph Wilson and Lucy Harrison (Gay) Wilson, was born in Richmond, Va., on January 15, 1882. He was a lineal descendant of Benjamin Harrison, of Virginia, signer of the Declaration of Independence, from whom also were descended two Presidents of the United States—William Henry Harrison and Benjamin Harrison.

Mr. Wilson received his early education in public and private schools of Richmond and Williamsburg, Va., and in Seven Islands Academy, Buckingham County, Virginia. He was graduated from Virginia Polytechnic Institute, at Blacksburg, Va., in 1906, with the degree of Bachelor of Science in Civil Engineering. He returned to the same institution as Assistant Instructor in Civil Engineering and Graduate Student, and was graduated with the degree of Civil Engineer, with highest honors, in 1907.

Mr. Wilson was then engaged for a year as Engineer with W. O. Burton and Company, Contractors on United States Government locks at Plaquemine, La. In 1908, he entered the employ, as Civil Engineer, of Winston and Company, General Contractors, of Richmond, Va., and Kingston, N. Y., then engaged on extensive contracts on the construction of the Ashokan Dam, aqueducts, and other projects connected with the water supply of New York, N. Y. In 1912 and 1913, he was a partner in the firm of Snead and Wilson on highway construction in New York State. Following this, Mr. Wilson was Managing Partner for about thirteen years with Winston and Company. During this period his construction work included 90 miles of highways in Maryland, West Virginia, Pennsylvania, and New York State, important projects on the Ulster and Delaware Railroad, in New York State, and the Chesapeake and Ohio Northern Railway, in Ohio, as well as bridges and miscellaneous engineering work.

In 1926, Mr. Wilson became affiliated with Winston Brothers Company of Minneapolis, Minn., under the firm name of Winston Brothers Company and

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[1] Memoir prepared by W. D. Maxwell, M. Am. Soc. C. E.

H. H. Wilson, of Harrisburg, Pa., this connection being continued during the remainder of his life. He was also President and Treasurer of the Lime Bluff Quarries Company; a Director in the All States Life Insurance Company; Director in the Peoples Bank, of Radford, Va.; Chairman of the Executive Committee of the Associated Pennsylvania Constructors; President of the Pennsylvania Construction Council; and had served for twelve years on the Board of Direction of the Associated General Contractors of America.

Mr. Wilson was perhaps best known as an exponent of the Construction Industry, being active in both State and National constructor associations. Following the entrance of the United States into the World War, Pennsylvania constructors engaged in the extensive State road-building program then under way, encountered difficulties brought on by embargoes on materials, increasing freight rates, mounting costs, and other unforeseen contingencies. These became so serious that members of the industry were threatened with business extinction. In this emergency Mr. Wilson, with other leading constructors, founded the Associated Pennsylvania Highway Contractors, later enlarged into the Associated Pennsylvania Constructors, a State-wide association of all classes of general constructors. His presentation of the constructors' case, his proffer of their complete co-operation with State authorities, and his exposition of how closely their interests are interwoven with the economic fabric of the State, were so impressive that the assistance of State Officials was secured, drastic relief efforts were instituted, and much permanent good was accomplished. Mr. Wilson served as the first President of the State Association, and upon his retirement, became Chairman of its Executive Committee, which office he held until his death.

Mr. Wilson was also one of the Founders and original Directors of the American Construction Council in 1922, at which time the Hon. Franklin D. Roosevelt was elected President of that organization. He was active in the Associated General Contractors of America, especially in its earlier years, and traveled thousands of miles and devoted much effective energy to the establishment of its many branches and chapters, while at meetings and conventions he was well known as an eloquent and effective speaker. As Chairman of its Committee on Contract Bonds and Insurance, he rendered notable service to the construction industry. His knowledge of this subject was extremely broad, and he was often called into consultation by surety companies and constructor organizations. He labored for years to secure more equitable provisions in public construction contract forms and succeeded in having many of his ideas incorporated in contract documents of State, county, and municipal public works departments.

On several occasions Mr. Wilson appeared before Congressional Committees, at Washington, D. C., to advise with them as to plans and policies affecting Federal construction. He attended a National Conference of the American Construction Council at Washington, in April, 1933, also a meeting of its Board of Governors which followed the Conference, and participated in drafting recommendations covering the National Public Works Construction Program which was submitted to President Roosevelt for action.

Exceptionally well informed and gifted with rare ability to translate movements and trends into definite statements and opinions, Mr. Wilson wrote exhaustively on contemporaneous construction matters and policies. For more than twelve years he acted as Editor-in-Chief of *Highway Builder,* and through that medium became well known to constructors throughout the Nation by reason of the clarity and excellence of his writings. He was a frequent contributor to *Engineering News-Record, The Constructor,* and other trade periodicals. Several years ago he published a series of articles in *Highway Builder,* under the pseudonym of "Monsieur X," on contract bidding methods which he later incorporated, with some additions and elaborations, into the manual, "Contract Cost Keeping for Highway Construction," a work which is still highly valued by the industry. He employed his leisure time for years in assembling genealogical data concerning early Virginia families, and at the time of his passing, was engaged in preparing "A History of Cumberland County" for publication.

Mr. Wilson was of a deeply religious nature; he was a baptized member of the Protestant Episcopal Church, and from childhood attended its services. He fostered reverence for patriotic ideals and found recreation in delving into early American history, his vacations often being spent in visiting shrines of historic interest. He maintained membership in the Virginia Historical Society, serving as its Vice-President; in the Hereditary Order of the Descendants of Colonial Governors; in the Dauphin County Historical Society; and in the Society of the Descendants of the Signers of the Declaration of Independence, serving for several years as its Secretary-General and, later, as its Second Vice-President-General.

He was married, in 1915, to Lily Tyler, daughter of former Governor J. Hoge Tyler, of Virginia, to which union were born three children, J. Hoge Tyler, Lily Norwood, and Henry Harrison, Jr., all of whom, together with his mother, survive him.

Mr. Wilson was elected an Associate Member of the American Society of Civil Engineers on November 26, 1918, and Member on July 16, 1928.

ROLLEN JOE WINDROW, M. Am. Soc. C. E.[1]

Died October 30, 1933

Rollen Joe Windrow, the son of C. C. and Mary Alice (Crenshaw) Windrow, was born at San Saba, Tex., on June 9, 1885. His early education and preparation for college was had in the public schools at Temple, and San Angelo, Tex. He entered the Agricultural and Mechanical College of Texas, College Station, Tex., in September, 1902, and was graduated therefrom with the degree of Bachelor of Science in Civil Engineering in June, 1906.

Upon the completion of his college course, Mr. Windrow entered the service of the Gulf, Colorado, and Santa Fé Railroad Company as Instrumentman on the construction of the Jasper Eastern Branch, between De Ridder and Oakdale, La., serving in this capacity through the completion of this line. He was also employed on the construction of the Texas and Gulf Railroad from Center to Gary, Tex. He then went to West Texas as Division Engineer on the construction of the Santa Fé Lines between San Angelo and Sterling City, Tex., and from Lometa to Eden, Tex.

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[1] Memoir prepared by E. L. Martin, Esq., Dallas, Tex.

In 1910, upon the completion of these lines, Mr. Windrow accepted an Instructorship in the Engineering Department at the Agricultural and Mechanical College of Texas, and pursued his engineering studies, majoring in Highway Engineering for the degree of Civil Engineer. Upon receiving this degree in June, 1912, he was appointed County Engineer of McLennan County, Texas, for the purpose of constructing a system of improved highways which was financed by one of the first major bond issues for this purpose in that State. His successful handling of this project no doubt did a great deal to promote similar projects over the State.

After completing the McLennan County Highway System, Mr. Windrow was appointed to the Board of Water Engineers of the State of Texas. He was a member of this Board for a short time only, as the City of Waco, Tex., called him to the position of City Manager. He was one of the first City Managers in Texas as the city-manager form of government was then in its infancy.

In 1919, upon the re-organization of the Texas State Highway Department, Mr. Windrow was appointed State Highway Engineer, and instituted an organization for the construction and maintenance of the State Highway System. His early experience in location, construction, and maintenance of railways was quite valuable to him as he patterned his staff very largely along the lines of railroad engineering organizations. His wide acquaintance with railroad engineers enabled him, even at that early date, to make great progress toward eliminating railroad grade crossings throughout the State.

In 1922, he was offered a position as Consulting Engineer for the Missouri Highway Commission by the Hon. Theodore Gary, Chairman of the Missouri Commission. Mr. Windrow accepted this position and organized the Missouri State Highway Department for the purpose of expending the large bond issue which had just been voted.

His ability as an Engineer and Executive had been recognized by Smith Brothers, General Contractors, of Dallas, Tex., and this Company made him a very attractive offer to associate himself with it, which he accepted in 1923. While allied with Smith Brothers Company Mr. Windrow supervised and handled some of the largest highway and municipal contracts in the Southwest. Among the more important and more interesting projects with which he was identified were the Kaw River Tunnel, at Kansas City, Mo., in connection with the water supply for that city; a tunnel and water-line for the Water Department of Fort Worth, Tex.; and similar work for Los Angeles, Calif.

In 1928, Smith Brothers Company withdrew to a large extent from the contracting field. Mr. Windrow disposed of his interest in that Company, and entered the gravel production and general contracting field on his own account, with headquarters in Dallas, in which he continued until his death on October 30, 1933.

Mr. Windrow was generally recognized by the Engineering and Contracting Professions in Texas and the Southwest as a wonderful leader of men, a capable organizer, and an able executive. He was likewise recognized as a most loyal and true friend by all those enjoying his friendship.

He was a Mason and a Shriner and was a member of the Dallas Athletic Club, the University Club, the Dallas Country Club, and the Rotary Club of Dallas.

He was married in 1917 to Sara Woodward Thompson, of Waco, and is survived by his widow, a son Rollen J. Windrow, Jr., and four daughters, Mary, Sara, Jean, and Jane Windrow. He is also survived by his mother, Mrs. M. A. Windrow, and three sisters, Mrs. Irene Purdom of Temple, Tex., Mrs. Alice McGalin, of Dallas, and Mrs. George Houghton, of Arlington, Tex.

Mr. Windrow was elected an Associate Member of the American Society of Civil Engineers on September 2, 1914, and a Member on April 21, 1920.

LOUIS PETER WOLFF, M. Am. Soc. C. E.[1]

DIED MARCH 8, 1933

Louis Peter Wolff was born at Red Wing, Minn., on December 14, 1867. He was the son of Henry and Amelia (Bottanes) Wolff. Mr. Wolff received his early education in the Grade and High Schools at Red Wing, graduating from High School in 1884. He attended the College of Engineering of the University of Minnesota in 1885 and 1886, but necessity required him to seek employment in railroad construction in April, 1886. He did not return to the University until the winter term of 1886-87, following which he again entered railroad work.

Early in 1886, Mr. Wolff became Resident Engineer in charge of seven miles of railroad construction for the Minneapolis, Lyndale, and Minnetonka Railway Company (now part of the Great Northern Railway). Beginning with the spring of 1887, Mr. Wolff was, successively, Resident Engineer in charge of construction on the Minneapolis, St. Paul and Sault Ste. Marie Railway and for the Duluth, Red Wing and Southern Railway Company.

In 1890, he became City Engineer of Red Wing, his birthplace, which position he held until 1899. In 1899, he was appointed City Engineer of Eau Claire, Wis. During these years, Mr. Wolff acquired an unusual understanding of the municipal problems of the cities and towns of the Northwest, and this early training provided a firm foundation for his later successful practice as a Consulting Engineer.

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[1] Memoir prepared by a Committee of the Northwestern Section consisting of George M. Shepard, and George H. Herrold, Members, Am. Soc. C. E., and Paul N. Coates, Assoc. M. Am. Soc. C. E.

During this early period of railroad and municipal engineering, Mr. Wolff met C. F. Loweth, Past-President, Am. Soc. C. E., then a Consulting Engineer, in St. Paul, Minn., and after a short period of joint consulting practice, he purchased the business from Mr. Loweth, in 1901, and continued in a general consulting engineering practice, with offices in St. Paul, from that time until shortly before his death.

Many of the early steel highway bridges still in use on highways in the Northwest, and several of the concrete bridges of later date, were designed by him and constructed under his direction. For several years he was Consulting Bridge Engineer for the Minnesota State Highway Commission. A thorough student and ever abreast of developments in matters pertaining to sewage disposal, he designed several of the earliest sewage disposal plants installed in Minnesota. It is interesting to note that, in later years, he was called upon to design additions to sewage disposal plants wherein parts of old septic tanks which he had designed many years previously were utilized and reconstructed to meet modern requirements.

Mr. Wolff was particularly successful in his plans for the development of water supplies. Among the cities in the Northwest for which new supplies were developed under his direction, or on which extensive improvements were made to the older ones, are included, Brainerd, Sauk Center, Red Wing, and Pipestone, Minn.; Eau Claire, Wis.; Mason City, Iowa; Glendive, Mont.; and Brookings, Milbank, and Huron, S. Dak. At Brainerd, Minn., following exhaustive research by Carl Zapffe, Consulting Geologist, and President of the Water and Light Board of that city, Mr. Wolff collaborated with him in the design of a manganese removal plant for the water supply. This plant has given very satisfactory results and will probably prove to be the pioneer development in the removal of this troublesome element from well-water supplies.

During the World War, Mr. Wolff offered his services to, and was appointed Consulting Engineer for, the United States Government in charge of design and supervision of construction for the water supply and sewage system for the large Army Cantonment at Camp Dodge, Iowa. For the making of valuations and appraisals of public utility properties, and as an arbitrator in such matters, the fairness and sound judgment of Mr. Wolff made his services much in demand.

In 1928, he was one of a Board of three engineers appointed by the City of St. Paul to make a valuation of the St. Paul City Railway Company, the findings of this Committee being accepted by the Railroad and Warehouse Commission of Minnesota.

Mr. Wolff took an active part in the public affairs of St. Paul and of the State at large. He was the first engineer on the State Board of Health, having been appointed in 1915 by Governor Hammond, and re-appointed by all succeeding Governors, until his retirement in December, 1930, due to ill-health. He was President of the State Board of Health and the oldest member in point of service when he retired. The records indicate that he never missed a meeting of that Board, or a meeting of the Executive Committee, during this period of fifteen years, except twice when he was absent from the city.

Mr. Wolff was a member of the St. Paul Association of Commerce, the St. Paul Athletic Club, and a Past-President of the Engineers' Society of St. Paul. His principal recreations were curling and chess.

From the time he came to St. Paul in 1901 he had taken an active part in the affairs of the various technical societies, both local and National. He gave freely of his time in furthering civic projects involving the interest of engineers. His intimate knowledge of municipal problems and his ability in passing on to others his sound conservative viewpoint and thoroughness made his advice much sought for, and wherever he was engaged his clients became his firm friends. Mr. Wolff was a man of retiring habits and of sterling character, and an untiring worker. He gave freely of his advice and help to the younger engineers with whom he was associated. Those of a younger generation who were so fortunate as to work with him find this training among their most valued assets.

He was married, in 1887, to Bertha DeGraff, of Dexter, Minn. Mrs. Wolff, a son, DeGraff, and a daughter, Mrs. Lyman Stanton, survive him.

Mr. Wolff was elected a Member of the American Society of Civil Engineers on June 30, 1910.

EDWARD MICHAEL BRENNAN, Assoc. M. Am. Soc. C. E.[1]

Died June 11, 1933

Edward Michael Brennan was born on November 29, 1887, at Sidney, N. Y., the son of Michael and Anna Brennan. He was graduated from Syracuse University, Syracuse, N. Y., in 1911, with the degree of Civil Engineer.

His first work was as Chainman in the Engineering Department of the Delaware and Hudson Railroad Company. He was then employed for two years in the Engineering Department of the New York Central Railroad Company, as Draftsman and Inspector on the construction of a new passenger station and station facilities at Utica, N. Y.

In October, 1914, Mr. Brennan entered the employ of the Public Service Commission of the State of New York as Junior Engineer on subway construction in New York City. Here he remained for two years in charge of field parties on the construction of the Seventh Avenue Extension of the West Side Interborough Rapid Transit Line, Routes 4 and 38, Section 2. This construction involved the underpinning of the Sixth Avenue Elevated Railroad and buildings adjacent to the subway structure, together with route and station layout.

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[1] Memoir prepared by F. B. Hank, Asst. Engr., N. Y. C. R. R., New York, N. Y.

In September, 1916, he returned to the service of the New York Central Railroad Company as Assistant Engineer at Ravena, N. Y. As such, he was in charge of field and office work in connection with the preliminary location surveys on the west side of the Hudson River for the proposed Castleton Cut-Off.

In November, 1917, Mr. Brennan, became a Designer in the Construction Department of the United States Navy. He was sent to the City of Santo Domingo, Dominican Republic, where he remained a year engaged in the design of docks, piers, and wharves.

In November, 1918, he again returned to the New York Central Railroad Company as Resident Engineer at Albany, N. Y. During the following four years, he was in charge of the Drafting Room of the District Office at Albany. The work handled during that time consisted of the termination of many contracts undertaken as World War measures and the negotiation of final settlements; the reconstruction of bridges; and other major maintenance items deferred during the war-time activity, and involving additions and betterments to existing facilities.

In May, 1922, the New York Central Railroad Company contracted for the construction of the Hudson River Connecting Railroad, involving fifteen miles of new location with yards, engine terminals, and accessory facilities. Mr. Brennan was transferred to this work as Assistant Engineer in charge of a section of the construction. During the next three years he supervised the construction of five miles of double-track line, including six highway-crossing elimination bridges; and the construction of two 30-stall engine houses, with power plant, coaling plant, ash-pits and turn-tables. Following the beginning of operation of the improvement, he continued for a year in charge of the construction of additional facilities. Mr. Brennan was then transferred to Harmon, N. Y., in charge of the construction of an engine house and additions to existing power-plant facilities.

In June, 1929, his services were loaned to the Toronto, Hamilton and Buffalo Railroad Company, with which Company he remained a year, in charge of the construction of an engine terminal at Hamilton, Ont., Canada.

In June, 1930, Mr. Brennan returned to the New York Central, but remained only six months when he again returned to the service of the Toronto, Hamilton and Buffalo Railroad Company as Construction Engineer in connection with a grade-separation project and the construction of a new station and office building, involving a cost in excess of $3 000 000. This work was commenced in the spring of 1931 and was about 95% complete at the time of Mr. Brennan's death. Here, again, he demonstrated his ability to handle works of this magnitude and maintain cordial relationships with the various interests with which he came in contact.

Mr. Brennan was a member of the Roman Catholic Church. He was married in 1912 to Florida G. Wilson, who survives him. He is also survived by a sister, Alice, and a brother, Raymond F. Brennan, both of Buffalo, N. Y., and a sister, Mrs. Edward Rogan, of Newark, N. J.

He was among the most promising of the Junior Executives of the Engineering Department of the New York Central Railroad Company. He possessed an unusual ability to make friendships, and was distinguished for his talent in securing co-operation and the highest grade of work from contractors. His untimely and sudden death is sincerely regretted by all who had the privilege of his acquaintance and association.

Mr. Brennan was elected an Associate Member of the American Society of Civil Engineers on April 16, 1918.

JOSEPH AUGUSTINE ALOYSIUS CONNELLY, Assoc. M. Am. Soc. C. E.[1]

Died January 1, 1934

Joseph Augustine Aloysius Connelly, the son of John and Mary (Riley) Connelly, was born in New York, N. Y., on August 15, 1881. He attended St. Monica's School and was graduated from Manhattan College, New York City, in 1902, with the degree of Bachelor of Science in Civil Engineering.

Mr. Connelly's first professional work, in the summer of 1902, was with the Northern Railway of Costa Rica, as Leveler on relocation. Returning to the United States, he served for a part of 1902 and 1903, as Leveler, with the Interborough Rapid Transit Railroad Company on the construction of the main power-house for the Rapid Transit System, in Manhattan.

On April 4, 1903, he accepted an appointment with the Rapid Transit Railroad Commission of New York City, and served as Rodman on the construction of the first rapid transit subway, being assigned to the construction of the loop of the Broadway Line in Battery Park, Manhattan, and the approach to the tunnel leading from Manhattan beneath the East River to Joralemon Street, Brooklyn, N. Y.

On August 14, 1905, Mr. Connelly accepted a transfer to the Department of Docks and Ferries, of the City of New York, where he was in charge of field parties making surveys for, and the construction of, piers, the taking of tide-gauge records, and the making of estimates. In April, 1906, he resigned to accept an appointment as Leveler in the Office of the Borough President of the Borough of Richmond, City of New York, where he was in charge of field parties on highway and sewer construction.

On May 11, 1906, Mr. Connelly again received an appointment by the Rapid Transit Railroad Commission, as Assistant Engineer, and was assigned to the construction of that part of the first rapid transit subway built in the Borough of Brooklyn, along Flatbush Avenue, from Nevins Street to Atlantic Avenue, where he was in charge of lines and grades.

The Rapid Transit Railroad Commission was succeeded by the Public Service Commission in 1907 and Mr. Connelly continued in its service. He was appointed Section Engineer and had charge of the construction of that part of the Fourth Avenue (Brooklyn) Line through Ashland Place and Fourth Avenue from Fulton Street to Sackett Street. On December 1, 1916, he was transferred from the Sixth to the Seventh Division of the Commission to take charge, as Section Engineer, of that part of the Seventh Avenue Line of the subway, along West Broadway, from Vesey Street to North Moore Street, Manhattan.

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[1] Memoir prepared by Eugene G. Haines and Aksel H. Jorgensen, Members, Am. Soc. C. E., and Robert F. Brushaber, Assoc. M. Am. Soc. C. E.

In August, 1918, Mr. Connelly resigned from the Public Service Commission to serve the United States Government on construction work at Columbia, S. C. Returning to New York at the end of the year, he was again appointed Section Engineer by the Public Service Commission and continued in the service of that Commission and its successors, the Transit Construction Commission, the Transit Commission, and the Board of Transportation, to the time of his death. He was in responsible charge, at different times, of the construction of a number of important sections of the subway systems of the City, including the inspection sheds and service buildings at the Jerome Avenue Yard of the Interborough Company; the construction of that part of the Eighth Avenue Subway Line of the Independent System along St. Nicholas Avenue from 123d to 132d Streets, in Manhattan; and that part of the subway line of the Independent System along the Grand Concourse and Boulevard from 161st Street to 167th Street, in The Bronx. The different contracts of which Mr. Connelly had responsible charge during construction, from time to time, had a contract value of about $15 000 000.

On the completion of the construction of the line along the Grand Concourse, Mr. Connelly was assigned to the Bureau of Contract Adjustment, where his familiarity with construction work and his past connection with many of the contracts was of great value to the Board of Transportation in contesting the numerous claims and suits filed against the City of New York and arising out of the construction of the subway system.

For more than thirty years Mr. Connelly served the city of his birth, faithfully and well, carrying easily the burdens and responsibilities placed upon him, unknown to thousands of those who benefit by the results of his labors. To those who were privileged to work with him, he will be remembered because of his fidelity to his work and his employers. He possessed in an unusual degree the ability to make friends, and they were legion. To these, there will remain the memory of his generosity, his unfailing good nature, and his absolute loyalty.

Gifted with a naturally fine singing voice he was always generous in its use for the pleasure of his friends. He was a member of the Roman Catholic Church and for thirty years sang at the Church of the Annunciation, at Convent Avenue and 131st Street, Manhattan; for the five years prior to his death, he had been soloist at Our Lady of Assumption Church, in Pelham Bay Parkway, The Bronx.

Mr. Connelly was married on June 8, 1908, to Margaret Seelig, of New York, who, with one daughter, Dorothy M. Connelly, survives him. He is also survived by his father, John Connelly, of New York City, and two sisters, Mrs. Isabel C. Carney, of Summit, N. J., and Mrs. Mary Broderick, of East Orange, N. J.

Mr. Connelly was elected an Associate Member of the American Society of Civil Engineers on February 4, 1913.

ALEXANDER SAMUEL DIVEN 3d, Assoc. M. Am. Soc. C. E.[1]

Died September 9, 1933

Alexander Samuel Diven was born in Elmira, N. Y., on May 23, 1893. He was the son of Eugene and Jeanette (Murdoch) Diven, and a great-grandson of General Alexander Diven. After preparing for college in the public schools of Elmira, he entered Lehigh University, at Bethlehem, Pa., from which he was graduated in 1915 with the degree of Civil Engineer.

After his graduation from college Mr. Diven entered the service of the Pennsylvania Railroad Company as an Engineer in the Maintenance-of-Way Department. He was engaged with this Company until January, 1917, being stationed in Elmira, New York, N. Y., and Baltimore, Md. He then entered the employ of David Lupton's Sons Company, in Philadelphia, Pa., where he served as Designer and Engineer until July, 1917.

From July, 1917, to June, 1918, Mr. Diven acted as Superintendent of Receiving for the American Red Cross, at Pier No. 1, New York City. In July, 1918, he enlisted in the United States Navy, receiving the rating of Chief Machinist Mate, and was assigned to duty at the Philadelphia Navy Yard, where he acted as Inspector in the Naval Aircraft Factory.

After his release from the Navy, in January, 1919, Mr. Diven was employed for a year on public utility appraisal for the firm of Forstall and Robinson, Consulting Engineers, in New York City. In January, 1920, he was appointed Office Manager for the American Society of Mechanical Engineers.

In September, 1921, Mr. Diven became Building Manager of the Engineering Societies Building and Accountant for United Engineering Trustees, Inc., in New York City, and continued in this position until his death. His duties included the control of the accounts for Engineering Foundation, Engineering Societies Library, and other organizations served by the Trustees. He was also Secretary of the House Committee for the Engineering Societies Building. He was active in the work of the Professional Engineers Committee on Unemployment, having had charge of the receipt and disbursement of funds for this work, for which United Engineering Trustees, Inc., is Treasurer.

Mr. Diven was held in high esteem by his associates and subordinates during his twelve years as Building Manager for the Engineering Societies. He was able and conscientious in his work and in attention to detail.

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[1] Memoir prepared by Gerald Staats Rinehart, Assoc. M. Am. Soc. C. E.

He leaves a widow, Marjorie (Carpenter) Diven, and a son, Eugene Liscum Diven. He was a member of the Alumni Association of Lehigh University, and a member of the Board of Governors of the Psi Upsilon Club, both of New York City.

Mr. Diven was elected an Associate Member of the American Society of Civil Engineers on December 28, 1931.

CHARLES STEWARD DONALD, Assoc. M. Am. Soc. C. E.[1]

DIED DECEMBER 9, 1932

Charles Steward Donald was born in Quincy, Mass., on August 27, 1890, the only son of William B. and Margaret S. (MacDonald) Donald.

He attended the John Hancock and the Willard Grammar Schools in Quincy until 1902, when he moved with his parents to Hardwick, Vt., where he completed his elementary education at Hardwick Academy, in 1907. In this same year he entered Coburn Classical Institute, in Waterville, Me., from which he was graduated two years later. He was very popular at Coburn and while there took a prominent part in several branches of athletics. In 1909, Mr. Donald entered Colby College, at Waterville, where he made the football, track, and baseball teams. While at Colby he was made a member of the Phi Delta Theta Fraternity, and Theta Eta Epsilon. In 1911, he entered the University of Vermont, at Montpelier, Vt., where he studied Forestry and played on the football team.

In 1912, Mr. Donald went to Winnipeg, Man., Canada, where he started work as a Rodman with a Survey Party, for the Grand Trunk Pacific Railway Company. He remained in the Canadian Northwest for the next three years and worked in various more and more responsible capacities in connection with the great railroad expansion which was being carried on at that time. During the last of these three years he was Resident Engineer for the Alberta and Great Waterways Railway Company; he traveled extensively over the entire Northwest and saw the beginnings of a great new country being opened to the White Man.

In 1916, Mr. Donald returned to the United States, and located in San Francisco, Calif., where he joined the Engineering Department of the Union Iron Works, and took a prominent part in connection with the extensive shipbuilding activities which this Company carried on during the World War. In 1918, at the expiration of the war, he joined the Southern Pacific Railway Company as an Assistant Engineer.

In 1921, he entered the employ of the City and County of San Francisco as an Assistant Inspector in connection with the construction of the O'Shaughnessy Dam. This dam, one of the largest in the world, forms the Hetch Hetchy Reservoir 170 miles from San Francisco, at the head of a stupendous development for supplying water to the City of San Francisco. Later, in this same year, Mr. Donald was made Resident Engineer and Superintendent of Construction at Priest Dam which forms another reservoir lower down on this same Water Supply System and which acts as a forebay for Moccasin Power-House. This plant utilizes the water on its course to San Francisco to generate electric power under a static head of 1 316 ft. In 1923, after the completion of the Priest Dam, he was appointed Resident Engineer in charge of construction of the penstocks for the Moccasin Power-House. He remained on this work until its completion in the latter part of 1925, when he returned to San Francisco.

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[1] Memoir prepared by James M. Renfrew, Esq., San Francisco, Calif.

Late in 1925, Mr. Donald joined the Engineering Department of the Board of Fire Underwriters, first as an Engineer experting earthquake insurance for buildings and, later, as Chief Engineer of the Pacific Sprinklered Risk Department, which position he held at the time of his death.

On December 9, 1932, while returning home in his automobile from Sacramento, Calif., after a business trip, Mr. Donald was struck by a train on a railroad crossing during one of the very rare snowstorms which occur in this section of California, and met an instant and untimely death.

On August 12, 1920, he was married to Maybelle L. Parker, of Alameda, Calif. He is survived by his widow; his mother, Mrs. Margaret S. Donald; and a sister, Mrs. Mary (Donald) Deans.

Mr. Donald was a member of Apollo Lodge No. 396, F. and A. M., and of the Board of Registration for Civil Engineers of the State of California.

He was a devoted husband and son, a staunch friend, and a Christian gentleman. His sincere nature and his keen sense of humor won for him a host of friends who will always remember him.

Mr. Donald was elected an Associate Member of the American Society of Civil Engineers on October 15, 1923.

ABRAHAM LEONARD DRABKIN, Assoc. M. Am. Soc C. E.[1]

DIED MAY 6, 1933

Abraham Leonard Drabkin, the son of Maurice and Pearl Drabkin, was born at Odessa, Russia, on June 28, 1888. He attended the public schools in the town of his birth, and laid the base for his future education by graduating from the Gymnasium (High School). The opportunities for a further education in those days did not come up to his expectations, and, in 1905, he left Russia for England where he lived for about four months.

On January 23, 1906, at the age of 18, Mr. Drabkin arrived in the United States. In the new land he found himself practically among strangers and without any visible means of support. He began his battle for existence, working at anything and everything that came his way, always with the thought of gaining a higher education as his goal.

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[1] Memoir prepared from information on file at the Headquarters of the Society.

He was not familiar with the English language when he reached these shores, but he already spoke French and German fluently. His knowledge of these languages aided him a great deal, and for two years he struggled studying English, at the same time trying to earn a living by working in paint shops, at a shoe polish factory, and in drug stores.

In 1908, Mr. Drabkin entered Cooper Union, in New York, N. Y., for the purpose of studying engineering. Receiving a scholarship from that Institution, he was graduated five years later, on May 29, 1913, with the degree of Bachelor of Engineering. In May, 1923, Mr. Drabkin received the degree of Civil Engineer from Cooper Union.

After his graduation, he joined the Engineering Forces of the Rapid Transit Commission of New York, as Draftsman in the Designing Department, and assisted in the construction of the Rapid Transit Subway. He held that responsible position for five years, from May, 1913, to July, 1918.

He was then appointed Designing Engineer with the firm of Wilputte Coke and Oven Company, in New York, where he also had the opportunity of displaying his engineering ability; he was with that Company from July, 1918, to March, 1919.

On June 16, 1919, Mr. Drabkin went to Detroit, Mich., and entered the service of the Ford Motor Company. Here, he was in complete charge of the design and construction of the power-house and foundry at the Ford River Rouge Plant. He left the employ of the Ford Motor Company on December 31, 1920, and entered the service of the City of Detroit as a Structural Engineer. He was instrumental in designing and supervising the construction of the Detroit City Garage.

By this time his ability in the profession had been recognized, and in 1921, he became associated with the firm of Albert Kahn, Inc., of Detroit, where for a period of more than ten years he was employed in the capacity of Structural Engineer. During his association with this nationally known firm, he aided in the design of such important buildings as the General Motors Building, Fisher Building, First National Bank Building, Griswold Building, Herman Kiefer Hospital, Ford Motor Company Hangar, and many others.

In 1929, a Russian Commission in search of architectural ability came to Detroit, and the firm of Albert Kahn, Inc., received the contracts for the design and erection of various buildings in Russia. On July 19, 1929, Mr. Drabkin left for Stalingrad, Russia, where, as Resident Engineer for Albert Kahn, Inc., until February 20, 1930, he successfully supervised the construction of the enormous Stalingrad Tractor Plant, the plans for which were designed in the Detroit Office of the firm, with the assistance of Mr. Drabkin. He completed this important work three months ahead of scheduled time, and returned to the United States on March 11, 1930. For the splendid services he rendered in connection with this plant, the Soviet Government made one of its rare exceptions by presenting him with a cash bonus in American money, also with a beautifully bound volume of the history of the plant, as well as a testimonial in Russian and in English, expressing the Government's appreciation and recognition of his talents as an engineer of high standing.

On his return to the United States, Mr. Drabkin resumed his work with Albert Kahn, Inc., and aided in the design of the Cheliabinsk Tractor Plant and the Upper Solda Structural Steel Plant for the Soviet Government. He was with the firm until June, 1932.

Mr. Drabkin was not content with being only an Engineer; he also studied pharmacy, and was graduated in June, 1923, with honors from the College of Pharmacy, in Detroit, as a Registered Pharmicist. In addition, having a rather versatile mind, he perfected many inventions, and patents for several of them were pending at the time of his death.

He was a fine linguist, familiar with the French, German, Spanish, and Italian languages, versed in literature and well read in general. He also found time for sculpture and painting, calling them his hobbies, and only a few months before he died he received honorable mention from an art school for a painting submitted by him. However, Mr. Drabkin's real hobby was writing poetry, and some beautiful specimens are scattered among his friends, in addition to the many in the possession of his family. Being of a very modest and reticent nature, he never had them published.

He died unexpectedly on May, 6, 1933, and is survived by his widow, Mrs. M. R. Drabkin. He is also survived by his father, a sister, and two brothers—one an engineer and one a physician.

Mr. Drabkin was elected an Associate Member of the American Society of Civil Engineers on June 4, 1928.

WILLIAM EDWARD FITZGERALD, Assoc. M. Am. Soc. C. E.[1]

Died February 9, 1931

William Edward Fitzgerald was born in New Brunswick, N. J., on October 18, 1889. He was the son of John Fitzgerald, who was a leader in civic affairs and held many public offices, and Angeline (Brooks) Fitzgerald.

Mr. Fitzgerald was graduated from the State Normal School, at Trenton, N. J., in 1908, and afterward went to Blair Hall, Blairstown, N. J., where he prepared for college. In 1909, he entered Lafayette College, at Easton, Pa., continuing his studies there until his Senior year, when the illness of his mother compelled him to leave college and seek employment, which he found in the office of the City Engineer of New Brunswick. After the death of his mother, Mr. Fitzgerald returned to Lafayette to complete his college course and was graduated therefrom in June, 1914, with the degree of Civil Engineer.

While in college, he took an active interest in all athletic sports and was a member of the football team at the time when Lafayette was producing some of the greatest football teams in its history; he had the satisfaction, therefore, of participating in many victories in the football field over rival colleges, He was also an active member of the Phi Gamma Delta Fraternity and helped greatly in making it an important factor in the life of the college.

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[1] Memoir prepared by Asher Atkinson, M. Am. Soc. C. E.

During his college years, Mr. Fitzgerald found employment in various capacities which would give him engineering experience. In 1911, he worked with the Union Electric Works, at Trenton, as a Draftsman.

After being graduated from Lafayette, he served for a short time with the Engineering Corps of the Pennsylvania Railroad Company, and, later, with the Interstate Commerce Commission on railroad valuation work. In 1916, he returned to the office of the City Engineer of New Brunswick and, for about three years, he was Principal Assistant Engineer, engaged on street paving, sewerage problems, and construction work for the Water Department. He had charge of the construction of an important dam, of the building of a Filtration Plant, and of necessary additions to the Pumping Station. After the Filtration Plant was completed, Mr. Fitzgerald acted as Superintendent for a short time, until he was assured that the plant was working satisfactorily. He then resigned to continue his engineering work in larger fields.

During the World War, from 1917 to 1919, he was able to lend valuable aid to the Government by assisting in the construction of two hundred houses, built to accommodate workers brought to New Brunswick, principally to work in the Wright-Martin Plant, building Hispano-Suisa motors for aeroplanes.

After leaving New Brunswick, Mr. Fitzgerald was employed in the Engineering Department of the United States Rubber Company and was stationed for a time at Naugatuck, Conn. In July, 1922, he was appointed Inspector of Construction during the building of the Delaware River Bridge between Philadelphia, Pa., and Camden, N. J., a work undertaken by the Delaware River Bridge Joint Commission. On November 21, 1922, he was promoted to be Assistant Engineer on this important work, and served in that capacity until the bridge was completed. After the bridge was opened for traffic, he continued there as an Inspector for the Maintenance Bureau of the Delaware River Bridge Joint Commission, and served in that capacity until his health compelled him to retire.

He was married on April 24, 1916, to Helen Coleman, of Bristol, Pa. Three children were born to them: John E., in 1917; Elizabeth, in 1919; and Ann, in 1925.

Mr. Fitzgerald had the type of mind which functions without apparent effort, and his quiet and efficient manner of going about his work won for him the confidence and respect of those with whom he came in contact, whether as superiors or subordinates. He was always personally popular, with warm human sympathies, and showed keenly his enjoyment of his contacts with close and intimate friends. This characteristic made him a welcome addition to any social gathering.

He was a devoted family man. He was unfortunate in that his elder daughter developed an obscure affliction which made it necessary for her to have constant care and medical treatment. He never wearied in his desire to do every thing that he could for her, and he was unremitting in his personal efforts to make life as easy and pleasant for her as possible, never permitting himself to doubt that she would eventually be restored to health. This daughter finally died of the disease from which she suffered so long, but her suffering was very much more easily borne because of the atmosphere of cheerfulness in which her father succeeded in keeping her.

Mr. Fitzgerald was elected a Junior of the American Society of Civil Engineers on May 31, 1916, and an Associate Member on April 14, 1919.

ANDREW VALERIAN GREAVES, Assoc. M. Am. Soc. C. E.[1]

Died May 9, 1933

Andrew Valerian Greaves, the son of Valerian E. and Nina (Kobyline) Greaves, was born at St. Petersburg, Russia, on December 27, 1901. He left Russia with his father during the Bolshevist Revolution, going to Siam where he remained for two years. At the age of nineteen, he was appointed an Engineer in the Royal Irrigation Department of Siam.

On leaving Siam, Mr. Greaves came directly to the United States, where he entered the Massachusetts Institute of Technology, at Cambridge, Mass., from which he was graduated with the degree of Bachelor of Science, with the Class of 1924.

After his graduation from college, Mr. Greaves was employed by Stone and Webster, Incorporated, as Tracer, until October, 1924. He then entered the employ of Vaughan Engineers, at Boston, Mass., as Instrumentman on preliminary surveys, stream flow studies, preliminary designs for water power developments, detailing and structural computations, etc., until January, 1926, when he was appointed Assistant Engineer in responsible charge of a wide range of work. He was also employed from 1929 to 1931, by Chase and Gilbert, Incorporated, in hydraulic construction and Court work.

In 1931, wanting to use to better advantage his natural talent for mathematical research, Mr. Greaves took up actuarial work with the Metropolitan Life Insurance Company, in New York, N. Y., where he seemed to be destined for a brilliant future Always enthusiastic in his work, he did not spare himself and, as a result of overwork, he suffered a serious nervous and mental breakdown, from which he never recovered He died at the State Hospital, at North Grafton, Mass., on May 9, 1933.

His pleasing personality won him many warm friends, and his keen mind and energy promised a successful scientific and business career. He was not married and is survived by his father.

Mr. Greaves was elected a Junior of the American Society of Civil Engineers on June 7, 1926, and an Associate Member on August 26, 1929.

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[1] Memoir prepared by John F. Vaughan, M. Am. Soc. C. E.

JOHN ALEXANDER GRIFFIN, Assoc. M. Am. Soc. C. E.[1]

DIED APRIL 12, 1933

John Alexander Griffin, son of David A. and Nancy Louise (Bush) Griffin, was born on August 26, 1880, at Lenoir, N. C. The Griffin family originally settled in Virginia, some of them moving later to Western North Carolina, among whom was the grandfather of John Alexander Griffin who acquired large land holdings in that section. John's father, David A. Griffin, like his father, was a successful farmer and a prominent man politically in his county, serving for years as Notary Public and Justice of the Peace.

John Alexander, one of thirteen children (nine of whom lived to maturity), grew up on his father's farm, and attended the schools in that section until his entry into the Agricultural and Mechanical College, at Raleigh, N. C., in 1900, where he began to realize that he had talents for Engineering and was influenced by his instructors to pursue that course of study.

Mr. Griffin began his interesting and successful career in the spring of 1902, when he entered the service of the Carolina and Northwestern Railroad Company as Rodman. From then, until his death, his unbroken career was as follows:

From February, 1903, to October, 1904, Instrumentman, Southern Railway Company, in charge of alignment, grade reduction, and double-tracking, during the last three months he had entire charge of seventeen miles; from November, 1904, to February, 1905, Leveler with the Tidewater Railroad Company. In February, 1905, he accepted a position as First Transitman with the Southern Railway Company, on location and on traversing and revising line. In February, 1906, Mr. Griffin received promotion to Assistant Engineer and was placed in charge of bridge work on branch lines near Donald, S. C., subsequently, having charge of alignment, grade reduction, and double-tracking at various places, and of construction of a branch line at Thomasville, N. C.; he also made surveys and assisted on the construction of the line between Monroe and Whittles, Va.

In January, 1910, Mr. Griffith was placed in charge of the Rivermont Tunnel at Lynchburg, Va., where he spent the following five years. From Lynchburg, he went to Norcross, Ga., in charge of double-tracking from Gainesville to Atlanta, Ga. Two years later he moved to Macon, Ga., as Chief Engineer, Maintenance of Way and Structures, with the Georgia, Southern and Florida Railway Company, in charge of lines from Macon, Ga., to Palatka, Fla. He held that position for about eight years and into the period of Federal Control of Railroads, during the World War, when he resigned to accept a position in Washington, D. C., with the United States

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[1] Memoir prepared by J. E. Teal, M. Am. Soc. C. E.

Railroad Administration. In this position, Mr. Griffin came in contact with railroad division officers of the United States Army and was awaiting a call for active field service in the World War at the time of the signing of the Armistice on November 11, 1918. His one regret in this connection was that conditions were not favorable to his induction into the Army at an earlier date.

While in Washington he accepted a position as Improvement Clerk with the Chesapeake and Ohio Railway Company, and moved to Richmond, Va., in 1924, reporting to the Assistant to the President. He was advanced to the position of Special Engineer in 1927, remaining in that capacity until his death.

He assisted in compiling statistics and factual exhibits in the Nickel Plate Unification Case in 1925 and 1926 and the Erie-Pere Marquette Case in 1927 and 1928, and in a number of other proceedings, before the Interstate Commerce Commission, in which the Chesapeake and Ohio Company was interested. Mr. Griffin's aptitude for research and analysis and his special experience in such cases qualified him for a service of particular importance in supervising the preparation of documents presented to the Interstate Commerce Commission in October, 1931, in support of the Four System Plan for consolidation of the railroads of the Eastern District of the United States.

Mr. Griffin's worth and his love for his work were attested by the fact that for three days only since his first position was he "without a job," and those days elapsed between his resignation from his position with the Georgia, Southern, and Florida Railway Company and traveling to Washington to accept the position with the Railroad Administration.

No task was too small to require his best efforts, consequently throughout his life he constantly was achieving worthwhile results and building the fine understanding character that he was. He was a man of calm but forceful personality; faithful and loyal to his family and friends, who not only gave him the esteem accorded rare gentlemen, but loved him dearly as well. As relaxation from his work his hobbies were golf and his home where he spent many happy hours working with his lovely flowers.

Religiously, the Griffin family leaned toward the Baptists and Methodists, and Mr. Griffin prized all his life a Bible awarded to him when he was a boy for faithful attendance at Sunday School over a period of years. He later adopted the Episcopal faith. He was a Mason, and while in Lynchburg, he was a member of the Hill City Lodge.

He was married on December 23, 1903, to Virginia Davis Grasty, daughter of John T. and Mary Elizabeth (Sale) Grasty, of Orange County, Virginia, who survives him.

The Engineering Profession has lost a member who thoroughly understood its magnificent attainments, and whose major ambition was to continue for many more years to contribute the knowledge gained from his varied experiences to its progress.

Mr. Griffin was elected an Associate Member of the American Society of Civil Engineers on January 3, 1911.

GEORGE HALVERSON, Assoc. M. Am. Soc. C. E.[1]

DIED APRIL 1, 1933

George Halverson, the son of Gunder and Bardina (Jorgensen) Halverson, was born at Kingston, N. Y., on June 9, 1891. He was educated at the public schools of his native town. In his youth Mr. Halverson learned the carpenter trade and wood carving and served as Carpenter Foreman on general construction, on heavy timber framing of bridges and buildings, and on residences.

In the meantime he was taking a correspondence course in Architectural Drawing, and attending night school, and from September, 1915, to June, 1917, he was a student at Pratt Institute, Brooklyn, N. Y., from which he received a Certificate in Architectural Construction.

In June, 1917, Mr. Halverson accepted employment with Stone and Webster Construction Company, in Boston, Mass., and was engaged in detailing reinforced concrete, and in general construction drafting and designing. He also attended the night course in Reinforced Concrete and Steel Designing at the Franklin Technical College, in Boston.

From January to June, 1918, Mr. Halverson was with the Thompson-Starratt Construction Company, of New York, N. Y., on general construction drafting for the United States Explosives Plant, at Nitro, Va., and from June, 1918, to February, 1919, he was with the United States Army Engineers as Master Engineer, Junior Grade (non-commissioned officer), with the 219th Engineers on engineering work and in training.

After his honorable discharge from the Army in February, 1919, he entered the contracting field at Kingston, N. Y., and was engaged in erecting houses, engineering design, drafting, etc., for George W. Betz, an Architect of that place.

In November, 1919, Mr. Halverson went to Portland, Ore., and until December, 1922, he was engaged in timber design and detailing on industrial buildings, beam and slab concrete highway bridges, heavy timber bridges and culverts, as well as dry dock construction and shop buildings for the Port of Portland and H. A. Rands, M. Am. Soc. C. E.

In January, 1923, Mr. Halverson removed to Los Angeles, Calif., where he was employed as Draftsman until October of that year by Norenberg and Johnson, Architects.

In October, 1923, he entered the service of the Union Pacific Railroad Company on general construction of buildings and culverts of reinforced concrete and steel and heavy timber designing. He was sent to the Headquarters of the Company at Omaha, Nebr., where he assisted in preparing the plans for the Terminal Shop Building and, later, returned to Los Angeles and was engaged on the Terminal Station at that place. Subsequently, he was made Superintendent and Inspector in charge of the construction of buildings for the Union Pacific Company.

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[1] Memoir prepared from information on file at the Headquarters of the Society.

From July, 1925, to March, 1926, Mr. Halverson was at Delray, Fla., engaged in contracting and the construction of commercial buildings, residences, etc. In April, 1926, he returned to Kingston, N. Y., and again entered the employ of Mr. Betz, where he remained until July, 1928. In this position he was engaged on the architectural drafting and engineering work in connection with the construction of private residences, commercial buildings, theatres, schools, churches, banks, etc. During part of this time, April to August, 1928, he worked nights on the engineering design of various jobs for Teller and Halverson, Architects, on foundations, reinforced concrete, steel and heavy timber trusses, etc.

From August, 1928, to January, 1929, he was employed by the Currier Engineering Corporation, of Newark, N. J., designing and detailing piping and machines, including the steel frames and foundations, for air-conditioning construction.

In March, 1929, Mr. Halverson entered the service of the firm of Teller and Halverson, and remained in its employ until July, 1931, except for the period from July to October, 1929, when his services were loaned to Mr. Betz to make plans and drawings for the addition to the Kingston High School Building. While with Teller and Halverson he was engaged on the design of steel and reinforced concrete construction and the making of architectural and construction details and preliminary drawings for a church, hospital, commercial store buildings, and residences.

From April to June, 1932, Mr. Halverson was again employed by Mr. Betz on plans and engineering designs for residences, etc. This proved to be his last engagement, for he was taken seriously ill in March, 1933, and died at the Kingston Hospital on April 1, 1933.

Mr. Halverson was not married, and is survived by his parents, Mr. and Mrs. Gunder Halverson, and a sister, Mrs. G. A. Craig, all of Kingston, N. Y.

He was a member of Hawthorne Lodge, A. F. and A. M., of Portland, Ore., and of the Society of American Military Engineers.

The following tribute to Mr. Halverson was received from R. L. Adamson, Chief Engineer of the Los Angeles and Salt Lake Railroad Company (Union Pacific System):

> "George [Mr. Halverson] and I were good friends, and during all the time I knew him, I never knew or heard of his doing anything except what was fair, upright, and honorable in every way. He certainly was a worthy man and a good friend."

Mr. Halverson was elected an Associate Member of the American Society of Civil Engineers on July 6, 1925.

HARVEY SYDNEY HENNING, Assoc. M. Am. Soc. C. E.[1]

Died December 17, 1932

Harvey Sydney Henning, the son of Walter S. and Lydia (Cunningham) Henning, was born at Ennis, Tex., on June 15, 1902. He received his engineering education at the Louisiana State University, Baton Rogue, La., during the years 1919 to 1923.

In the spring of 1923 Mr. Henning worked as Instrumentman on highway location for the Louisiana State Highway Commission. From July, 1923, until late in 1924, he was with the Louisiana State Board of Health as Assistant Sanitary Engineer, engaged on water supply and sewerage inspections and supervision of malaria-control field work.

He left the Louisiana State Board of Health in 1924 to accept a position as Assistant Parish (County) Engineer in De Soto Parish, Louisiana, where he was in charge of plans and specifications for Parish highways, and supervised construction.

From January, 1928, until his death Mr. Henning worked with various consulting engineers, engaged in the design and supervision of construction of water supply and sewerage systems and general municipal improvements. At the time of his death, he was associated with the Henry A. Mentz Company, Incorporated, Consulting Engineers, and was just about to put into operation a water treatment plant and distribution system, designed and constructed under his supervision for the East Jefferson Water-Works District No. 1 (Jefferson Parish, Louisiana).

Mr. Henning's professional career, although brief, covered a wide range of activity and demonstrated ability of high degree.

He was an active member of the Louisiana Section of the Society and of the Masonic Order.

Mr. Henning is survived by his parents, his widow, the former Margaret Clarke, of Baton Rogue, La., to whom he was married on June 23, 1924, and two children.

His passing is felt as a distinct loss by his colleagues who admired and respected his professional qualifications and integrity; and by a host of friends who loved him for his personal charm.

Mr. Henning was elected a Junior of the American Society of Civil Engineers on July 15, 1929, and an Associate Member on February 3, 1931.

ANDREW PETER LUDBERG, Assoc. M. Am. Soc. C. E.[2]

Died April 11, 1934

Andrew Peter Ludberg was born on June 29, 1889, at Ostersund, Sweden. His parents were Andres and Martha Logdborg, and he was the fourth of five children. When he was four years old, the family came to America, settling in Polk County, Wisconsin, where they acquired citizenship, and the present Americanized spelling of their name was adopted. Some years later, they moved to Moscow, Idaho, and there Andrew Ludberg attended the preparatory school conducted by the University of Idaho and completed two years of collegiate work in the University. Returning to Wisconsin, he finished his course in the Engineering Department of the University of that State, and was graduated in 1911 with the degree of Bachelor of Science in Civil Engineering. High achievements in scholarship gained for him membership in the National Engineering Honor Fraternity of Tau Beta Pi.

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[1] Memoir prepared by John H. O'Neill, Assoc. M. Am. Soc. C. E.

[2] Memoir prepared by Albert F. Reichmann, M. Am. Soc. C. E.

After his graduation Mr. Ludberg served as Concrete Inspector at Madison, Wis., and, later, as Detailer of reinforced concrete structures for the Chicago, Milwaukee, St. Paul and Pacific Railroad Company, in Chicago, Ill.

From 1913 to 1921, Mr. Ludberg was employed as Structural Draftsman by the American Bridge Company, at Gary, Ind. In 1921, he returned to the University of Idaho as Associate Professor of Civil Engineering, and taught the courses relating to structures. In 1923, the University of Wisconsin honored him with the degree of Civil Engineer. Returning to Gary in 1927, he re-entered the employ of the American Bridge Company, remaining until 1928 when he accepted the position of Chief Draftsman in charge of the Chicago Office of the Lackawanna Steel Construction Corporation. From there, he went, in January, 1934, to assume the duties of Resident Engineer in charge of the steel superstructure on the Little Bay Bridge, near Portsmouth, N. H., where he was employed at the time of his death.

While engaged in his usual morning round of inspection of the bridge, Mr. Ludberg fell accidentally, and was instantly killed. His funeral was held on April 14, 1934, in the City Church, at Gary, Ind., and interment followed in that city.

Mr. Ludberg was a member of the City (Methodist Episcopal) Church, of Gary, a member of the Official Board of the church, and a Director of Sunday School work and of boys' athletics. He found recreation in golf and tennis, and played the latter game with ability far above the average.

He was a member of the Western Society of Engineers and of the Society for the Promotion of Engineering Education. In his undergraduate days, he became a member of the Idaho Chapter of Beta Theta Pi Fraternity. He was a frequent contributor on technical subjects to the *Idaho Engineer.*

His traits of character, his personality, and his intellect won for Mr. Ludberg the respect and admiration of all who knew him. Of a quiet and unassuming nature, he possessed mental ability of high order. Particularly remarkable was his skill in mathematical analysis and insight into the elastic behavior of structures, especially those of the "higher" and indeterminate types. Coupled with his theoretical knowledge was a finely balanced judgment and sense of the practical which enabled him to deal with all classes of structural work in a manner, adequate, efficient, and economical. In his passing, his home, his friends, and his profession have suffered a great loss.

On August 3, 1912, he was married to Everetta Bass, of Montello, Wis., also a graduate of the University of Wisconsin and a fellow classmate.

Wherever they made their home, they were happy in giving devoted service to their church and community. Their children, who with Mrs. Ludberg survive him, are Alice, James, and Beth.

Mr. Ludberg was elected an Associate Member of the American Society of Civil Engineers on August 4, 1924.

HERBERT LAWRENCE LUTHER, Assoc. M. Am. Soc. C. E.[1]

Died September 23, 1933

Herbert Lawrence Luther was born at Lawrence, Kans., on October 27, 1885, and grew to manhood in that city. His parents were Caleb Manning and Sarah Elizabeth (Lawrence) Luther.

His early education was received in the Grade and High Schools of Lawrence. His professional training was obtained at the University of Kansas, at Lawrence, from which he was graduated in June, 1909, with the degree of Bachelor of Science in Civil Engineering.

In his vacations previous to graduation from the University, Mr. Luther was employed by the Atchison, Topeka and Santa Fé Railway Company as Rodman and Inspector and by the El Paso and Southwestern Railway Company from the time of his graduation until September, 1909.

He then entered the service of The Missouri Valley Bridge and Iron Company, of Leavenworth, Kans., and continued with this firm until the time of his death. From September, 1909, to August, 1916, he was employed as Assistant Engineer at the Home Office on designing, detailing, and estimating bridges, both superstructure and substructure.

In August, 1916, Mr. Luther went to Boise, Idaho, as Branch Manager of the Western Office of the Company, where he contracted for a number of bridges over important streams.

In October, 1924, the office in Boise was closed, and Mr. Luther went to Dallas, Tex., as Branch Manager of the Southwestern Office, where he continued until the time of his passing. His duties in Dallas were of a nature similar to those in Boise, and he had many good structures to his credit.

In his work he contacted many engineers and contractors, and wherever he went he made close and lasting friendships. His genial, friendly manner endeared him to his co-workers and competitors alike. It was not his way to speak ill of any one; if he could speak no good, he remained silent.

To his firm he was loyal and faithful, always industriously looking after its interests. He was an industrious, capable engineer, willing to share his experience with, and advise, younger men when desired.

While at college Mr. Luther was a member of Alpha Tau Omega Fraternity and, later, joined the Benevolent and Protective Order of Elks, and the

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[1] Memoir prepared by E. H. Barkmann, M. Am. Soc. C. E.

Masonic Order, as well as the Kiwanis and High Twelve Clubs. At the time of his death, he was a member of the Scottish Rite Temple, at Boise, Idaho, and the Shrine Temple, at Dallas.

He was married on October 19, 1910, to his college sweetheart, Frances A. Houlton, who survives him, together with their two daughters, Arlene and Frances. Mr. Luther enjoyed an ideal home life. He was a devoted husband and father, and his first concern was the welfare and happiness of his loved ones.

Mr. Luther was elected an Associate Member of the American Society of Civil Engineers on May 31, 1916.

FRANCISCO XAVIER MEMIJE, Assoc. M. Am. Soc. C. E.[1]

DIED DECEMBER 22, 1932

Francisco Xavier Memije was born in Manila, Philippine Islands, on October 8, 1889, the son of Gervacio Memije and Consolacion (Vivencio del Rosario) Memije.

He was graduated from the University of the Philippines in 1913, with the degree of Bachelor of Arts; and, in 1926, received the degree of Master of Science in Civil Engineering from the University of Santo Tomas, at Manila.

Mr. Memije was one of the earliest Filipinos to practice civil engineering in the Islands. He entered the service of the Insular Bureau of Public Works in November, 1913, as Junior Assistant Engineer. Late in 1914, he resigned this position to enter the employ of the Mindoro Development Company as Assistant Engineer on the location and survey of its irrigation project. In February, 1915, however, he returned to the Bureau of Public Works as Junior Civil Engineer. Later, he became Assistant Civil Engineer, and then District Engineer of Pampanga Province.

In August, 1918, he transferred to the Manila Railroad Company as Assistant Engineer, subsequently being appointed Project Engineer in the Maintenance-of-Way Department, which position he held at the time of his death. Mr. Memije gave special attention to architectural design and was identified with the design and construction of the special stations and modern structures of the Manila Railroad.

He was a member of the Philippine Institute of Engineers and Architects, in the work of which he was active.

In 1914, Mr. Memije was married to Dionisia Estrada, of Calasiao, Philippine Islands, who survives him. He will be remembered by his friends and associates, to whom his sudden death was a shock, for his sterling character, extreme patience, and helpful nature.

Mr. Memije was elected an Associate Member of the American Society of Civil Engineers on January 16, 1928.

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[1] Memoir prepared by H. V. Campbell, M. Am. Soc. C. E.

WILLIAM ARNOLD NEWMAN, Assoc. M. Am. Soc. C. E.[1]

Died October 30, 1933

William Arnold Newman was born in Jessamine County, Kentucky, on August 20, 1871, the son of James William Newman and Susan (Arnold) Newman. His early education was obtained in the Jessamine County Schools. He attended Transylvania College, at Lexington, Ky., from which, in 1893, he obtained the degree of Civil Engineer. From 1893 to 1896, Mr. Newman was an Instructor in Chemistry at Transylvania College and also a Special Student at the University of Kentucky, in the same city.

He began his engineering career as Instrumentman for the Cincinnati, New Orleans, and Texas Pacific Railroad Company, and was employed by that Company from 1896 to 1899.

In 1899, Mr. Newman accepted a position as County Surveyor of Fayette County, Kentucky, which position he retained until 1901. From 1901 until 1906, he served as Engineer in Charge of Location and Construction for the Blue Grass Traction Company, later known as the Kentucky Traction and Terminal Company. While thus employed, forty-five miles of traction line were constructed under his personal supervision.

By this time Mr. Newman had acquired a reputation for engineering ability and was employed from 1907 until 1910 by the City of Lexington, as Superintendent of Public Works.

In April, 1910, he again entered railroad service as Division Engineer of Maintenance for the Louisville and Nashville Railroad Company. He was transferred to Engineer of Construction of the Eastern Kentucky Division of this line in 1913, and remained in this capacity until October, 1915.

From 1915 until 1917, Mr. Newman was engaged in consulting engineering work in the City of Lexington. In the latter year he was appointed by the University of Kentucky as Assistant Professor of Civil Engineering. He served as Assistant Professor until 1926, and was then made a Professor of Railway Surveying and Geodesy. He continued teaching until October, 1933, at which time he was confined to his bed with supposedly only a minor ailment. However, his condition grew steadily worse, and he died on October 30, 1933.

Professor Newman was affectionately known as "Daddy" Newman to all his engineering students and was as truly interested in their welfare, as if each had been his own son. He was an authority on land surveying, particularly in Fayette County and the adjoining counties. He knew the land owners and their boundaries, and his opinion and work were seldom questioned. He was a man of fine character and wielded a large influence on the lives of all students who came in contact with him. It was his practice to obtain various kinds of work in the city and county for members of the Senior Civil Engineering Class. Not only was he known and liked in the College of Engineering, but he imparted a bit of romance and color to the other colleges on the University Campus.

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[1] Memoir prepared by D. V. Terrell, M. Am. Soc. C. E.

Professor Newman was a member of the Christian Church, and of the Independent Order of Odd Fellows. He was elected to Honorary Membership in Tau Beta Pi in 1926.

He was married, on December 8, 1897, to Mary Stuart, who survives him, together with three children, his mother, one brother, and three sisters.

Professor Newman was elected an Associate Member of the American Society of Civil Engineers on March 15, 1926.

HERBERT PATERSON, Assoc. M. Am. Soc. C. E.[1]

Died January 7, 1934

Herbert Paterson was born in Edinburgh, Scotland, on August 17, 1865. He was the youngest son of Dr. Robert Paterson, who, at that time, was President of the College of Physicians and Surgeons, of the University of Edinburgh. Dr. Paterson was a noted physician of his day, and was called in to attend King Edward VII of England, when the latter was stricken with typhoid fever.

Herbert Paterson received his elementary education in private schools in Scotland and England. Later, he entered the University of Edinburgh to study medicine under the direction of his father. After pursuing a medical course for three years, he became convinced that the field for his talents was not in medicine, but in engineering. The colleges of England and Scotland had no course in engineering at that time, and the only way to secure technical instruction was to serve an apprenticeship. Accordingly, it was necessary for him to enter into what was called an Indenture of Apprenticeship, with Messrs. Brockbank, Wilson, and Mulliner, Civil Engineers, of Manchester, England. The agreement drawn up at that time is an impressive legal document. It is written entirely by hand and specified that, Herbert Paterson must serve for five years, during which time he is to govern himself as a good apprentice should. He shall not disclose any of the secrets of his employers, nor steal anything from their office. His father shall provide him with "good and sufficient meat, drink, washing, lodging, wearing apparel, and all other necessaries," and, in addition, pay the firm One Hundred Guineas. All these conditions were fulfilled, and Mr. Paterson completed his apprenticeship in 1887.

His first position was with John Waddell and Sons, Contractors, of Edinburgh. This firm was at that time engaged in the construction of the Mersey Tunnel, a submarine vehicular tunnel beneath the Mersey River, near Liverpool, England. This type of work was unusual in those days, and the experience gained by Mr. Paterson proved valuable to him in later years. The tunnel was completed in 1888.

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[1] Memoir prepared by J. L. Paterson, Esq., Long Beach, Calif.

Mr. Paterson was then engaged by Reed and Campbell, Contractors, of London, England. This firm had contracted the construction of a drainage system for the Valley of Mexico, the main unit of which was what is known as the Tequixquiac Tunnel. Because of his previous experience in this type of work, Mr. Paterson was sent to the City of Mexico as Assistant Engineer. Three years later he was made Chief Engineer and served in this capacity until the completion of the project in 1893. He was always considerate of his employees and made it a rule never to ask a man to undertake any dangerous work that he would be unwilling to perform himself. Because of this, he gained the love and respect of the natives. The importance of the project to the City of Mexico commanded the attention of President Porfirio Diaz. It was inevitable that he and Mr. Paterson should meet, and a friendship immediately developed between them. President Diaz entertained extensively for him at the Presidential Palace. It was with many regrets that Mr. Paterson finally took his leave to return to his home in Scotland.

After visiting his home for approximately one year, Mr. Paterson could no longer resist the urge to visit other lands. He went to Canada, where he became Construction Engineer for the Winnipeg Street Railways, in Winnipeg, Man., holding this position until 1895. Mr. Paterson then became interested in mining, and secured the position of Manager of the Mikado and Elizabeth Mines, which were located on the shores of the Lake of the Woods, in the Providence of Ontario, Canada. At that time Canada was sparsely settled and the facilities for transportation were very poor. Some of Mr. Paterson's fondest recollections were of the days when he made trips between these mines and the Town of Rat Portage (now Kenora) a distance of 80 miles, in canoes, with Indian guides. He often told of the difficulties he encountered in mining, concentrating, and smelting the ore in these primitive surroundings, and of the wild game that had never before seen human beings, as evidenced by its utter lack of fear of men. In this work, his training in medicine was invaluable. There being no physician at the mines, it was often necessary for him to give medical treatment and care for injuries to his men.

In 1898, Mr. Paterson returned to Winnipeg, to become Examiner of Surveys for the Province of Manitoba. He held this position until 1902, when he resigned to open offices for the private practice of his profession.

About that time his youngest daughter became seriously ill, necessitating a change to a warmer climate. Mrs. Paterson and the children went to Southern California and took up their residence in Pasadena. One year later, the daughter having apparently recovered, they returned to Winnipeg. This proved to be a bad move, however, for the daughter became ill again shortly thereafter, and Mrs. Paterson again returned to Southern California, settling in Long Beach. This daughter died in 1914.

In 1905, Mr. Paterson closed his offices in Winnipeg and joined his family in Long Beach. Shortly thereafter he accepted a position with the

Southern California Edison Company, making preliminary surveys for power plants and transmission lines. Many of the lines of transmission towers in Southern California stand as monuments to mark the path of his footsteps.

In 1908, Mr. Paterson resigned this position to become Vice-President and Chief Engineer of the Canadian Investments of California, Incorporated, a firm at that time engaged in the development of fruit lands and an irrigation system in the San Joaquin Valley, near Porterville, Calif. This proved to be an unprofitable venture, however, so in 1910 he returned to Southern California to enter the office of the Los Angeles County Flood Control Commission, where he remained approximately two years.

In 1914, Mr. Paterson was appointed Hydraulic Engineer for the City of Long Beach. A rapidly growing town, Long Beach was in need of additional sanitary facilities, and his work consisted mainly in designing sewerage systems.

In 1918, Mr. Paterson received an offer from the Phelps, Dodge Corporation to become Divisional Engineer for its copper mines at Pilares de Nacozari, Sonora, Mexico. This position he held for two years, when he became Chief Engineer. During his association with this Company, he did much to standardize timbering in this particular mine, which to that time had been rather a haphazard affair. He also tried to introduce the use of standard forms of reinforced concrete for this purpose. From his former experience in the City of Mexico, and his fluent command of the Spanish language, he became on very good terms with the native population and local authorities, and on several occasions averted trouble for his employers from those sources.

In 1921, Mr. Paterson again returned to Long Beach, Calif., to become Sanitary Engineer for that city. In this capacity he designed practically all the sewers, storm drains, and underground pumping plants in the city. By this time he had become very much interested in Sanitary Engineering, and through practice, private study, and University courses, became proficient in the work. The sanitary system of the City of Long Beach, through his efforts, became a model for other cities of the Pacific Coast, and remains as an everlasting monument to him.

In 1928, Mr. Paterson was appointed Assistant City Engineer of the City of Long Beach, and, in 1929, became City Engineer. In 1930, he was made Assistant Director of Public Service and City Engineer.

In his work as a City official, Mr. Paterson never lost sight of the principle he laid down for himself, and which he stated as follows: "The people of the City of Long Beach have placed me in this position to look after their interests. As long as I remain here I will do so with all my power." He gained a high reputation and the respect of all with whom he came in contact for his honesty and integrity. He guarded zealously the trust that had been placed in him, and no political influence or possibility of personal gain ever influenced his recommendations, or caused him to vary from the soundest engineering practice. For this, he was admired by those who knew him and understood his principles. During his term as a public official he was known as a conserver of the taxpayers' money. Innumerable savings in their behalf remain in the records of the City of Long Beach.

Being of a retiring disposition, never thrusting himself forward, and modest to the extreme, Mr. Paterson won hosts of friends by his sincerity and his devotion to his duties. His many responsible positions were gained through sheer engineering ability. His reputation for honesty and integrity is well demonstrated by the confidence that was placed in him. He had an analytical and retentive mind. He was well read and, consequently, extremely well informed on practically every subject that was broached to him. He was always a student, and, no matter in what branch of engineering he might be engaged at the moment, was always adding to his store of knowledge along other engineering lines.

During his lifetime, Mr. Paterson had many hobbies at various times, among which were photography, commercial art, engraving, assaying, cabinet making, model ship building, etching, painting, and chemistry. Even these hobbies he followed wholeheartedly, and managed to find time to take courses of study pertaining to them. These hobbies developed an inventive turn of mind, and he had patented various devices along a mechanical line.

In his youth he had received an excellent musical training. He played the violin magnificently, at one time being first violinist in the Church of the Holy Name, in Manchester, England. This was considered quite an honor, and a recognition of musical ability.

That he endeared himself to his associates and superiors alike is best demonstrated by the following statement, issued at the time of his death, by the employees of the Engineering and Public Service Departments of Long Beach:

"Pat has taken the long trail. We like to remember him as having worked up to the last. Happy must be the man who carries on to the last. Only yesterday he was busy here in the office, doing his work like a veteran, always on time, always busy. To-day he is gone. He went from us suddenly. His memory will be fresh with us many years."

The following resolution was passed by the City Council of Long Beach, Calif.:

"*Whereas,* There comes a time in the life of every man when he must be his own Columbus, called upon to pass down into that undiscovered country from whence no traveler has ever returned to tell of its wondrous beauties: of its streets paved with gold, beside which flows a river of solid silver, upon whose banks there grow trees whose leaves are for the healing of the nations; and,

"*Whereas,* the City of Long Beach has recently been called upon to record the passing of one of its foremost and most respectable citizens in the person of Mr. Herbert Paterson, whose long period of unselfish service to the City of Long Beach, as an employee, is now and always has been recognized as exemplary and high grade in every particular;

"*Therefore Be It Resolved,* by the Mayor and City Councilmen of the City of Long Beach, that they express in this manner their high opinion of Mr. Paterson, both personally and as an employee, to the bereaved wife and relatives, and express to them their deepest sympathy in their affliction caused by his passing. * * *."

In 1889, Mr. Paterson was married to Alice Maud Sanderson, the daughter of Charles Horace Sanderson, of Leeds, England, and the granddaughter of

Charles Wetherell Wardle, one of the founders of Manning, Wardle and Company, of Leeds, England, which firm was for many years a leading manufacturer of contractors' locomotives. They had four children; Mrs. Paterson and three of the children survive him. They are, Mrs. Gwendoline May Wilson, John Lionel Paterson, and Robert Charles Paterson. The fourth child, Anita Claire, preceded her father in death by twenty years.

Mr. Paterson was elected an Associate Member of the American Society of Civil Engineers, on August 31, 1925.

RICHARD FRANKLIN REY, Assoc. M. Am. Soc. C. E.[1]

Died October 7, 1933

Richard Franklin Rey, the son of Herman F. and Carrie Rey, was born in Diamondale, Mich., on November 17, 1898. He was educated in the pubilc schools of Lansing, Mich., and, later, attended and was graduated from Michigan State College, at East Lansing, Mich., in June, 1922, receiving the degree of Bachelor of Science in the College of Mechanical Engineering. In June, 1927, he also received a Professional Degree in Civil Engineering from Michigan State College.

In the summer of 1922, Mr. Rey began his engineering career as a Draftsman and Instrumentman with the City Engineering Department of Lansing. He adapted himself very well to municipal engineering work and soon afterward was placed in charge of all bridge construction and maintenance as City Bridge Engineer. Some of the projects built under his design and supervision include: Shiawassee Street Bridge and grade separation; Kalamazoo Street Bridge and grade separation; Saginaw Street Bridge; Grand River Avenue Bridge; and the Logan Street Bridge and grade separation. All these structures are of concrete construction.

In July, 1933, Mr. Rey accepted a position with the Michigan State Highway Department as Engineer in Charge of Bridge Maintenance. He was holding this position at the time of his death.

Although Mr. Rey was only 34 years old when he died, he had accomplished that which would do credit to a much older man. He was quiet and reserved, and was well thought of by all who knew him.

He took an active part in the affairs of the Lansing Engineers Club, and was a Registered Civil Engineer in the State of Michigan. He was a member of Blue Lodge No. 33, F. and A. M., of Lansing.

He was not married and is survived by his father and a brother, Clarence.

Mr. Rey was elected an Associate Member of the American Society of Civil Engineers, on October 26, 1931.

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[1] Memoir prepared by Harold B. Huntley, Esq., Lansing, Mich.

JAMES HORNER RICE, Assoc. M. Am. Soc. C. E.[1]

Died August 8, 1933

James Horner Rice, the son of Patrick J. and Nora B. (Parker) Rice, was born in Little Rock, Ark., on December 20, 1887. His father was an Engineman and Conductor on the Missouri Pacific Railroad, and ran the first engine over the line into Pine Bluff, Ark., in 1878, and the first full passenger train into Little Rock, in 1880. His mother, a native of Chicot County, is a lineal descendant of one of the most cultured and aristocratic of Southern families. She is a member of the Arkansas Pioneers Society, the Daughters of the American Revolution, Daughters of 1812, the United Daughters of the Confederacy, and other civic and patriotic organizations.

James Horner Rice was educated at the Arkansas Military Academy, at Little Rock, the Christian Brothers College, at St. Louis, Mo., and the University of Arkansas, at Fayetteville, Ark., where he majored in Civil Engineering, under the late Julius James Knoch, Assoc. M. Am. Soc. C. E., and Dean W. N. Gladson, later Vice-President of the University of Arkansas. During his University course, from September, 1906, to February, 1909, Mr. Rice made a most brilliant record as a student.

From February, 1909, to March, 1911, he served as Installation Engineer for London Hill, Consulting Engineer, of Little Rock, on streets and roads. From March to June, 1911, he was Assistant Engineer on the location of the Little Rock, Missouri and Western Railway, and from June to December of the same year, he served as Resident Engineer on concrete pavements then being laid in Little Rock, involving 48 000 000 sq yd.

In December, 1911, Mr. Rice was appointed Chief Engineer of the Little Rock, Missouri and Western Railway Company, and served in this capacity until February 1915. As such, his work included the construction of 38 miles of new road. He was then appointed Location Engineer with the State Highway Department of Arkansas, under the Hon. W. B. Owen, State Highway Commissioner, which position he held until May, 1915.

From May, 1915, until October, 1917, Mr. Rice served as Construction Engineer with Dickinson and Watkins, Consulting Engineers, of Little Rock, on asphalt roads and drainage. He also supervised the laying of 160 000 sq yd of asphalt road surface by the penetration method. From November, 1917, until May, 1919, he was engaged as Assistant Supervising Engineer of Camp Pike, in charge of topographic surveys, water supply and sewer construction, roads, railroad connections, and asphalt streets. As World War Governor of Arkansas, it was the writer's privilege to recommend Mr. Rice for this position, which his achievements as an Engineer fully warranted.

From May, 1919, to April, 1925, Mr. Rice was Superintendent of Public Works of the City of Little Rock, and had charge of the maintenance of 120 miles of pavement, the operation of an asphalt plant with an output of 900 000 sq yd, involving a system of pavements constructed over a six-year period.

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[1] Memoir prepared by the Hon. Charles Hillman Brough, Little Rock, Ark.

He was the Senior Member of the firm of Rice-Knott, Consulting Engineers, of Little Rock, from May, 1925, to March, 1928, and, in this capacity, designed and constructed 350 000 sq yd of pavement; 28 miles of sewer; investigated Kentucky rock asphalt deposits, near Covington; constructed 11 000 lin ft of sea walls at West Palm Beach, and Sarasota, Fla.; and directed the appraisals of four large public utilities for rate-making purposes. He was also frequently called upon to give expert testimony before the Federal Courts of various Southern States.

From March, 1928, to November, 1930, Mr. Rice served as Construction Engineer on asphalt pavements for the Arkansas State Highway Department, collaborating with the Van Trump Testing Laboratory, of Chicago, Ill., on the revision of asphalt specifications for the Highway Department. From November, 1930, to June, 1931, he was Consultant Contractor on the compilation of cost data and bids on asphalt pavements, in Arkansas, Louisiana, and Texas, and from June to December, 1931, he was Treasurer of the Colonial Marble Company, of Rutland, Vt. In 1933, he rounded out his brilliant career as a Civil Engineer by serving as Asphalt Engineer with the Arkansas State Highway Department.

Mr. Rice was married on June 21, 1916, to Marguerite English, granddaughter of a former Justice of the Supreme Court of Arkansas, and a daughter of P. D. English, Clerk of the Supreme Court for more than a quarter of a century. This union was blessed by three sons—James, Peyton, and Pat Joe—and a daughter, Marguerite.

In addition to his widow and children, Mr. Rice is survived by his mother, Mrs. P. J. Rice, and two sisters, Mrs. Tom Harding and Mrs. Elmer A. McCaskill, all of Little Rock, and a brother, Walter Rice, of Memphis, Tenn.

He was a member of the Benevolent and Protective Order of Elks, Little Rock Lodge No. 29, Little Rock Engineers Club, and the Dog Island Hunting Club. Funeral services were held at his home, in Little Rock, on August 9, 1933, and his remains were interred in Calvary Cemetery. James Horner Rice was a cultured Christian gentleman of the old Southern school.

Mr. Rice was elected an Associate Member of the American Society of Civil Engineers on October 8, 1918.

ROYDEN KARL SCHLAFLY, Assoc. M. Am. Soc. C. E.[1]

DIED MAY 24, 1916

Royden Karl Schlafly was born at Mt. Eaton, Ohio, on September 17, 1878. His boyhood was spent in Mt. Eaton where his early education was obtained. He entered the Ohio State University, at Columbus, Ohio, in the fall of 1899 and was graduated with the degree of Civil Engineer from that institution in 1903.

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[1] Memoir prepared by F. H. Eno, M. Am. Soc. C. E.

After his graduation, Mr. Schlafly was employed by the City of Columbus as an Assistant Engineer. In 1904 and 1905, he served as Resident Engineer on the construction of the Scioto River Dam, later named the Griggs' Dam. This structure, the cost of which was $348 000, is 1 100 ft long, of concrete, and impounds the waters of the Scioto River for a water supply for Columbus.

In September, 1905, Mr. Schlafly was employed by the Ohio State University as Assistant in Civil Engineering. He was a conscientious, hard-working man, capable, keen in analysis, indefatigable in the performance of duty, always willing in service. While in the University employ, he perfected the methods of scheduling courses of studies, far beyond anything accomplished in that line before his time.

He was promoted from Assistant to Instructor and, later, to Assistant Professor, which position he held until 1914, when he resigned to accept a position with the Ohio State Highway Department. In June, 1915, he was transferred to the State Bureau of Inspection and Accounting, as an Examiner of Construction, which position he held at the time of his death.

In 1907, Mr. Schlafly was married to Ruth Richards, of Columbus. To them were born two children, Ida Louise, on January 30, 1913, and Roy Karl, Jr., on January 2, 1917—after Mr. Schlafly's death.

He was highly esteemed by all who knew him, by his professional associates, his associates at the University, and by his neighbors and friends. He was a member of St. Paul's Reformed Church, of Mt. Eaton, Ohio.

Mr. Schlafly was elected an Associate Member of the American Society of Civil Engineers on September 3, 1913.

HERBERT DAVIS SCHUTT, Assoc. M. Am. Soc. C. E.[1]

DIED JANUARY 10, 1934

Herbert Davis Schutt was born in Chesaning, Mich., on July 24, 1888, the only son of John W. and Alice (Davis) Schutt. He was a graduate of the High School, at Elmira, N. Y., where he lived for a number of years. He was one of the Honor Graduates of Union College, at Schenectady, N. Y., in the Class of 1909, when he received the degree of Bachelor of Engineering in Civil Engineering.

After his graduation from college, Mr. Schutt entered the private practice of engineering and, until January, 1910, was engaged on surveys, building estimates, etc. For about eighteen months, from January, 1910, to June, 1911, he was employed as Foreman and Superintendent, and later, as Traveling Supervisor, for the Mohawk Engineering and Construction Company, of Schenectady, in connection with contracts for New York State Highways and the Barge Canal. From June, 1911 to September, 1913, Mr. Schutt had charge of surveys and engineering work for A. P. Patrick, Dominion Land Surveyor, of Calgary, Alberta, Canada.

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[1] Memoir prepared from information on file at the Headquarters of the Society.

In September, 1913, he entered private practice in Calgary, and until November, 1914, was extensively engaged in the design of reinforced concrete structures, etc. Afterward, until May, 1915, he was employed on surveys, building estimates, etc.

From May to October, 1915, Mr. Schutt held the position of Assistant Engineer in the Department of Parks, in Schenectady, and, as such, had charge of laying out and constructing roads in Central Park in that city. Later, he designed and was the Contractor for a reinforced concrete dam built by Mr. William Goodale, at West Chazy, N. Y.

From January, 1916 to July, 1918, Mr. Schutt was associated with the General Fireproofing Company as Traveling Engineer on the design of structures and estimating quantities. He also supervised the sales and promotion work of the Company. In July, 1918, he was appointed Field Engineer with Dittoe, Fahrnestock, and Ferber, Architects and Engineers, of Cincinnati, Ohio, and until June, 1919, was in charge of engineering design and supervision of construction.

Mr. Schutt was then made Steel Superintendent of the Thomas McShaffey Company, of Akron, Ohio. In this position which he held until July, 1920, he was responsible for the reinforcing steel layouts, with general supervision of the work on the extension of the B. F. Goodrich Plant, in Akron. He then went to Cleveland, Ohio, as Steel Superintendent for Arthur G. McKee Company, for which Company he had charge until July 1921, of steel layouts, reinforced concrete design, and general supervision of reinforced work.

From July, 1921, to March, 1924, Mr. Schutt was employed by the City of Schenectady, and until December, 1923, was an Assistant Engineer acting in a supervisory capacity on six of the Public School buildings. He was also in charge of various paving projects for the City. In January, 1924, he was made First Assistant City Engineer in charge of estimates for and construction of city improvements.

In May, 1924, he was made District Engineer of the Fireproof Products Company of New York, N. Y. As such, he was engaged in the design of structures, the making of estimates, etc. About the same time he served as Construction Engineer for the City of Schenectady on damage claims.

At the time of his death, Mr. Schutt was the Owner and Manager of the Fire-Proofing Materials Company, Incorporated, with offices in Schenectady. He had served as a member of the Code Committee of the City of Schenectady and, a short time before his death, he was put in charge of all the engineering work in Schenectady which was planned under the supervison of E. W. Allen, Chairman of the Civil Works Administration in that city. Mr. Schutt had been connected with this work only three days when he was taken ill. The position to which he had been appointed was in connection with one of the most valuable projects in Schenectady's relief program, and owing to his close association with the City Engineer's Office, his services were valuable to the work.

While living in Calgary, Alberta, Canada, from 1911 to 1914, Mr. Schutt served as First Sergeant, G Company, of the 103d Regiment of the Canadian Militia. He had been active in rifle clubs both in Canada and in Schenectady and was regarded by his friends as a good rifle shot and an authority on rifles and pistols.

He was a member of the American Association of Engineers, the Professional Society of Engineers of New York State, and was a Licensed and Professional Engineer and Land Surveyor in New York State.

In addition to active membership in the Junior Order of United American Mechanics, Mr. Schutt was a member of St. George's Lodge No. 6, F. and A. M., St. George's Chapter 157, Royal Arch Masons, and St. George's Commandery 37, Knights Templars. He was also an active member of Gehomecobe Grotto, Mystic Order of the Veiled Prophets of the Enchanted Realm, and a Past Patron of George Hope Chapter, Order of the Eastern Star, which office he had held for five years. He was a member of Sigma Xi Fraternity, and previous to joining the Union Street Christian Church, had been an active member of the Emmanuel Baptist Church, of Schenectady.

Those who knew Mr. Schutt well, regarded him as a hard-working engineer and one whose qualifications were the admiration of his fellow craftsmen.

He was married in October 10, 1911, to Ethel Chadwick, the daughter of Mr. and Mrs. John T. Chadwick, of Schenectady. In addition to his widow, a daughter Betty Schutt, and a son, J. Donald Schutt, survive him.

Mr. Schutt was elected an Associate Member of the American Society of Civil Engineers on April 20, 1928.

GILBERT COBB STAEHLE, Assoc. M. Am. Soc. C. E.[1]

DIED FEBRUARY 6, 1934

Gilbert Cobb Staehle was born at White Bear, Minn., on March 19, 1889, the son of George Henry and Harriet Morrison (Cobb) Staehle. He received his preliminary education in the Grammar and High Schools of St. Paul, Minn. His professional training was interrupted by his employment, but he was graduated from the University of Minnesota, at Minneapolis, Minn., with the degree of Bachelor of Science, in 1920, and Master of Science in Civil Engineering in 1922. Mr. Staehle was granted the degree of Civil Engineer by the University of Minnesota, in 1931, for a thesis and research on the elastic properties of concrete. He was elected a member of Sigma Xi the same year.

Mr. Staehle was employed as a Draftsman with the Tri-State Telephone and Telegraph Company, St. Paul, in 1908 and 1909. In 1909 and 1910 and, again, in 1911, he worked for the Minnesota, St. Paul and Sault Ste Marie

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[1] Memoir prepared by Frank S. Altman, Dist. Engr., Portland Cement Assoc., Minneapolis, Minn.

Railway Company ("Soo Line"), as Draftsman and on surveys for new construction from Frederick to Superior, Wis., and of town sites on the Cuyuna Iron Range. In 1910 and 1911, he was engaged as a Draftsman with the late Max Ernst Robert Toltz, M. Am. Soc. C. E., Consulting Engineer, of St. Paul, on the design of the steel and concrete ore dock of the Great Northern Railway Company, at Superior, Wis., preliminary plans for the Union Station and Terminals, at St. Paul, as well as highway and railway bridges and other structures.

From 1911 to 1913, Mr. Staehle served as Draftsman and Designer for the Trussed Concrete Steel Company, of Detroit, Mich. (later, the Truscon Steel Company, of Youngstown, Ohio), on the design of reinforced concrete structures, including the Presidential Palace, at Havana, Cuba, reservoirs in Hawaii and the Philippine Islands, and other structures in the United States, Mexico, and South America. From 1913 to 1915, he was Chief Draftsman and Chief Engineer for the Concrete Engineering Company, at Omaha, Nebr., designing reinforced concrete foundations and frames and structural steel frames for a number of large buildings in Chicago, Ill., Detroit, Cleveland, Ohio, Boston, Mass., Los Angeles, Calif., and other large cities.

From 1915 to 1921, Mr. Staehle was District Engineer at St. Paul and Minneapolis for the Truscon Steel Company and designed numerous large reinforced concrete structures in the "Twin Cities," Duluth, and Chicago. He was Consulting Engineer on the design of St. Mary's Hospital, Rochester, Minn., which contains a three-story amphi-theater and several lesser amphitheaters, at which Dr. Charles Horace Mayo, Dr. William Benson Mayo, and other famous surgeons perform operations, and which involved unique structural problems. He also designed a hollow reinforced concrete dam of unusual height.

From 1921 to 1925, he was Consulting Engineer at Minneapolis, specializing in reinforced concrete. He designed the reinforced concrete frame for the $1 500 000 Library at the University of Minnesota, the $800 000 High School, at Chisholm, Minn., a highway and electric railway bridge with three main spans of 300 ft each, and a railway bridge of unusual design. From 1925 to 1927, he was Consulting and Research Engineer, for the Barton Steel Company, at Chicago, and in general consulting practice. From 1927 to 1929, he was with the Bridge Department of the Chicago and Northwestern Railway Company and designed numerous railway and highway bridges of steel, concrete, and timber, also the foundations for the twenty-story Merchandise Mart, in Chicago, which has air rights over the Chicago and Northwestern tracks.

From 1929 to 1931, Mr. Staehle was Associate Engineer, with the Structural and Technical Bureau, of the Portland Cement Association, at Chicago. In this position, he was in charge, with F. E. Richart, M. Am. Soc. C. E., of the reinforced concrete column investigation of the American Concrete Institute at the University of Illinois, which comprised designing, making, curing, and taking strain measurements of several hundred columns and about 2 000 companion control cylinders, and included an investigation of effects of fast, slow, and sustained loading, elastic properties, shrinkage, and plastic flow phenomena for concrete strengths ranging from 2 000 to 8 000 lb per sq in. His work also included investigation of the practicability of long-span reinforced concrete arches for a number of large bridge projects in the western part of the United States and the development of design for rigid frame and other structural problems.

As Senior Engineer of the Bridge Department of the Indiana State Highway Commission, from 1931 to 1933, Mr. Staehle designed a number of steel, concrete, and timber structures, including a steel cantilever bridge of 500-ft main span, and a seven-span reinforced concrete arch bridge with unusual substructure features.

In 1933, with G. A. Maney and Charles A. Ellis, Members, Am. Soc. C. E., Mr. Staehle prepared designs for a proposed high bridge over St. Louis Bay, between Duluth, Minn., and Superior, Wis., having a 1 100-ft main span, 135 ft above high water, and approaches ½ mile long at each end.

Mr. Staehle was the author of a number of engineering papers, including a discussion of "Plain and Reinforced Concrete Arches"[2]; discussion of "Plastic Flow in Concrete Arches"[3]; "Reducing Specific Weight of Concrete"[4]; a discussion of "Concrete Arches for Long Span Construction," by E. Freyssinet[5]; and problems in Chapter V of "Statically Indeterminate Stresses," by John I. Parcel and G. A. Maney, Members, Am. Soc. C. E. Mr. Staehle also checked the "Moment and Shear Charts for Continuous Beams and Rigid Building Frames"[6] prepared by Norman M. Stineman, Assoc. M. Am. Soc. C. E.; and was co-author with Professor Richart of "Progress Reports of Reinforced Concrete Column Investigation at University of Illinois."[7]

Original in all his work, Mr. Staehle developed new methods and revolutionized many systems of design and methods of handling work. He was credited with having put the computation of reinforcing steel on a high standard of design and economy. He invented three systems of centering and patented one, and developed new types of concrete floor construction and movable bridges. He was a man of rigid honor, great mental refinement, reserved, yet unobtrusively conveying quiet authority. He had a keen appreciation of music and architecture, enjoyed photography, and was a serious student of history and political economy. He had a profound love for the woods and forests, and liked to take long trips into the wilderness. He was kind and considerate of others, and never voiced criticism or censure.

He was married, at Chicago, Ill., on October 26, 1912, to Agnes Jean MacMahon, who survives him.

Mr. Staehle was elected a Junior of the American Society of Civil Engineers on December 3, 1913, and an Associate Member on November 27, 1917.

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[2] *Journal*, Am. Concrete Inst., October, 1932.

[3] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 709.

[4] *Civil Engineering*, June, 1932.

[5] *Loc. cit.*, February, 1932.

[6] *Journal*, Am. Concrete Inst., January, 1933.

[7] *Loc. cit.*, 1931 and 1933.

PAUL STEENSTRUP, Assoc. M. Am. Soc. C. E.[1]

Died May 16, 1933

Paul Steenstrup was born at Larbrofos, near Kongsberg Silver Mine, in Norway, on August 20, 1880. He was the son of Peter Steenstrup, a Major in the Norwegian Army, and also a Civil Engineer and Manager of the Labro Wood Pulp Mills. His mother was Marie (Kielland) Steenstrup.

The boy's early education was gained in a good home environment and in good schools. When he was about 15 years old he was admitted to, and later was graduated from, the Royal Navy Academy of Norway, where he received an engineering education as well as naval training. He made several cruises as a Naval Cadet, but he did not enjoy the restrictions and drilling, and resigned when he was about 18 years of age.

Before his untimely death, of cancer, Major Steenstrup had admonished his wife that she should advise all their children to go to their uncle (the writer), who was in the United States; so eventually three sons and one daughter came to this country. The oldest, Peter, arrived in 1895.

After resigning from the Navy, Paul Steenstrup was engaged as an Overseer on a large sugar plantation in British Guiana, South America. Here were gathered a company of adventurers, the sons of many nations, engaged to watch Hindoo laborers on time contracts in the sugar-cane fields. These men could be compared with the French African soldiers of the Foreign Legion. The Sugar Company provided them with food and lodging. They policed from horseback; they gambled, fought, boxed, wrestled, and drank much. When Mr. Steenstrup was 19 years old he contracted malarial fever and dysentery. Being ordered to leave that climate, he came to New York, N. Y., in 1899.

He found his brother, Peter, in a New York hospital and there he stayed also, with malaria. When he had recovered his health sufficiently, for a period of ten years he held many positions all over the United States. He was a fair-minded, just man; a man possessed of good humor, a keen sense of loyalty. and unlimited resourcefulness. He accepted correction from his superiors, but from others he would tolerate no interference; he could never serve two masters. He weighed only 160 to 170 lb, but was as quick as a steel trap and any pretender who crossed him once never tried to give orders to, or to interfere with, Paul Steenstrup thereafter.

He became an American citizen in 1904, and obtained a certificate as Construction Engineer from the International Correspondence Schools, at Scranton, Pa., in order to become better versed in American methods and terminology. Before 1910, Mr. Steenstrup served as Assistant Engineer on construction and location of railroads, and was employed on the New York Central, the Buffalo Susquehanna, the Buffalo Creek, the Northern Pacific, the Chicago, Milwaukee and St. Paul, and other railroads. From 1910 to 1923 his activities were mostly in private practice, as Consulting Engineer and Contractor in the States of Washington and Oregon. At one time during this period he was Assistant County Engineer, at Olympia, in Whatcomb County, Washington.

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[1] Memoir prepared by S. T. M. B. Kielland, M. Am. Soc. C. E.

His subsequent activities may be recorded, as follows: From 1923 to 1925, Assistant Engineer on the Hetch-Hetchy Water and Power Projects, in charge of tunnel lining at Groveland, Calif.; from 1925 to 1927, Assistant Engineer with the Interstate Bridge and Tunnel Commission, on the Holland Vehicular Tunnel, under the Hudson River, New York City, in charge of field work at the Jersey City, N. J., approaches; in 1927 and 1928, Office and Field Engineer of Jacobs and Davies, Incorporated, Consulting Engineers, New York City, in charge of field work and designs, and Supervisor of sea-wall construction, reclamation, etc.; and from 1929 to 1933, he was almost exclusively connected with the California Highway Division as Resident Engineer in District 1. He died rather suddenly from heart trouble, in Eureka, Calif.

Paul Steenstrup's career resembled that of thousands of others—with a difference. Consider a young, well educated man trying to find a place in the world; he has ideals, but practical work introduces him to life that is not ideal—or its ideals differ from his plans. In this sense Paul Steenstrup was an extraordinary person, considering his many positions and his many changes of location. First, there appears a "cocky" young gentleman in a trim uniform, a cavalier, and a cadet of the Royal Navy School. Severe training and discipline interfere with his actions; all freedom is regulated and while it was interesting to visit foreign countries on cruises, parading and being entertained, the prospect of success and advancement is limited, and he decides to resign and go to British Guiana, about as undesirable a place as could be found.

Paul Steenstrup's forefathers make a remarkable background for an engineering career. About 1800 his great-grandfather, Paul Steenstrup (whose name he bore), was Chief Engineer and President of the celebrated Kongsberg Silver Mines. His wife's brother was a Government cabinet member. They both were delegates to the great National Convention of 1814 and signers of Norway's Constitution.

Peter Steenstrup, his grandfather, was a Naval Officer and Mechanical Engineer. He founded the first, and the most successful, industrial shipbuilding establishment of its time in Norway. His father was both a Civil Engineer and a graduate of Norway's Military Academy, corresponding to West Point in the United States. He was also successful in railroad construction as Engineer and Manager; he built one of the leading wood pulp mills in the country. All of Mr. Steenstrup's brothers were engineers and on his mother's side there were at least eight chief engineers, covering all branches. A very extraordinary family of engineers this—Civil, Military, Mining, Mechanical, Electrical—including at least ten Chief Engineers. Tracing back 200 years one finds several mining engineers, ancestors of this family, from Saxony, Germany, who came to Norway and married. What influence had these conditions on the career of Paul Steenstrup? His successes and his failures, his mental processes and his physique—were they due to inheritance or to specialized education and environment?

On September 24, 1907, Mr. Steenstrup was married to Minda Selmer Graff, of Seattle, Wash., daughter of the late Johannes Graff who, at one time, was Chief Engineer of a railroad in British Columbia. Her mother was a sister of the one-time Premier Minister, Selmer, of Norway. There are two daughters, Ida and Ruth, by this first marriage. He was married again on October 19, 1926, to Ellen May, of Dresden, Saxony, a former war nurse.

Paul Steenstrup was misunderstood by many, which was mostly their own fault. His mind was keen, enlightened, and inspiring. As an Engineer, and a polytechnic, he was exceedingly able. To the writer, he was a dear friend, a good comrade, a noble and most generous gentleman.

Mr. Steenstrup was elected an Associate Member of the American Society of Civil Engineers on October 1, 1926.

LEIGH E. STEVENS, Assoc. M. Am. Soc. C. E.[1]

Died November 5, 1933

Leigh E. Stevens, the son of FitzEdward and Ida May (Parker) Stevens, was born on May 27, 1879, in Paw Paw, Mich. Here, he grew to manhood and received his early education.

Following his graduation from High School, Mr. Stevens went to work for the Pere Marquette Railroad Company as Rodman and, later, as Leveler, thus earning a portion of his tuition before entering the University of Michigan, at Ann Arbor, Mich. He was obliged to give up his college work at various times and was employed as Draftsman and Inspector of Pavements for the City of Dowagiac, Mich.; on drainage and survey work for Cass County, Michigan; and, later, with the United States Engineers, at Grand Rapids, Mich.

In 1907 Mr. Stevens was graduated from the University of Michigan with the degree of Bachelor of Science in Civil Engineering, going immediately to Ambridge, Pa., where he began work in the Bridge Department of the American Bridge Company under the late Rolla G. Manning, M. Am. Soc. C. E., who was also a native of Southern Michigan.

From 1908 to 1909, Mr. Stevens was City Engineer at Dowagiac, serving in that capacity for one year when he went to Grand Rapids where he was employed as Draftsman and Estimator in the City Engineer's Office. The following year he was made Chief Draftsman, and, later, Assistant City Engineer. In 1912, he became City Engineer. This position he held until 1915, when he went to Sioux Falls, S. Dak., where he entered private practice, in which he continued for three years.

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[1] Memoir prepared from information on file at the Headquarters of the Society.

In 1918, Mr. Stevens went to the Depue, Ill., Plant of the New Jersey Zinc Company, working in the Drafting-Room and Maintenance Department. In 1921 and 1922, he was employed by the Receiver of the Kalamazoo, Lake Shore, and Chicago Railway Company on valuation and accounting, and in the Bridge Division of the Missouri State Highway Department. He then returned to the employ of the New Jersey Zinc Company, at Depue, where he remained until his sudden death, from coronary thrombosis, on November 5, 1933.

On August 10, 1912, at Dowagiac, he was married to Genevieve Howser who, with two sisters and a brother, survives him.

Mr. Stevens was a member of the Masonic Fraternity and its auxiliary, the Order of the Eastern Star. He was of quiet and studious disposition, and was never satisfied until he had exhausted his subject. His sense of honor was of the highest. He was especially fond of young people; he enjoyed their company and was interested in their welfare and advancement. His books were presented to the University of Michigan Engineering Library.

Mr. Stevens was elected an Associate Member of the American Society of Civil Engineers on February 6, 1912.

IVAN FORREST WHITE, Assoc. M. Am. Soc. C. E.[1]

DIED SEPTEMBER 22, 1933

Ivan Forrest White, the son of Richard and Anna (Bell) White, was born on a farm near Harrisonville, Mo., on October 22, 1882. His boyhood and summer vacations while in college were spent on this farm, where he developed a liking for gardening, which he retained all his life. He studied Civil Engineering in the University of Missouri, at Columbia, Mo., from which he was graduated in 1906 with the degree of Bachelor of Science.

After graduation Mr. White went to work for the Southern Pacific Railroad Company in Tucson, Ariz., as a Computer of earthwork. He continued in this position until 1909, when he was employed by the Kansas City Terminal Railway Company as Draftsman on track layout for the Union Station. In 1913, he moved to Los Angeles, Calif., where he was employed by the Pacific Electric Railroad Company as a Draftsman on right-of-way maps and work-order sketches for the Maintenance-of-Way Division. After a few months with the California Highway Commission, Mr. White accepted a position, in 1915, with the United States Indian Irrigation Service. His first assignment was to the office in Salt Lake City, Utah, plotting topographical maps and designing irrigation structures. In 1917, he was transferred to the Los Angeles Office, where he had charge of the preparation of reports of operation, as well as the making of maps and designs for new construction.

In October, 1922, Mr. White became Deputy City Engineer of Venice, Calif., and as such he supervised the preparation of plans and estimates, and drew up the ordinances for street improvements, storm drains, and sanitary sewers for that city. For a short time during 1924 he acted as City Engineer. In 1925, Venice was annexed to the City of Los Angeles, but the City Engineer's Office was continued as a branch of the Los Angeles City Engineer's Office, and Mr. White was retained as Assistant District Engineer, acting as Office Engineer, preparing reports, and supervising the design of numerous improvements. In 1930, he became District Engineer, a position he held at the time of his death.

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[1] Memoir prepared by H. K. Palmer, M. Am. Soc. C. E.

Mr. White was painstaking and thorough in all his work, and when in municipal offices, was ever mindful of the best interests of the taxpayers. He was well liked by his co-workers and had many true friends.

His early life on a farm gave him a love of the out-of-doors and, as a boy, he was fond of hunting and fishing which, in later life, took the form of camping. He liked nothing better than week-end automobile trips, and his summer vacations were spent on long tours or camping in the mountains. At home, he delighted in keeping up his garden and making improvements about the house.

In 1908, he was married to Edna Grace Morgan. Mrs. White and a son survive him.

Mr. White was elected an Associate Member of the American Society of Civil Engineers on October 14, 1930.

OLIVER EARLE YOUNG, Assoc. M. Am. Soc. C. E.[1]

Died July 6, 1933

Oliver Earle Young, a son of Dr. Oliver C. and Margaret E. (Cannedy) Young, was born in Gaston, Ala., on March 15, 1886. He received his Grade and High School education in York, Ala., from which schools he was graduated in 1901. In the fall of that year he enrolled in the Alabama Polytechnic Institute, at Auburn, Ala., from which he received his degree of Bachelor of Science in Mining Engineering in 1905. He was a member of Phi Kappa Alpha Fraternity and throughout his four years at the Institute he was held in highest esteem both by the Faculty and his fellow students and was the recipient of many honors from his classmates who knew him "as a military officer to be very strict" but "a mighty fine fellow," and as one of the "stern men with empires in their brains," and as "a man of undoubted honor."

Mr. Young's first engagement in his chosen profession was as a Rodman, during his summer vacation in 1903, for the Cuba Railway Company, at Camaguey, Cuba. After his graduation, from June to September, he was an Assistant Engineer in the Maintenance-of-Way Department of the Pennsylvania Railroad Company.

From September, 1905, to February, 1906, he was Assistant Resident Engineer, and from February to August, 1906, Resident Engineer, for the Schloss-Sheffield Steel and Iron Company, on coal and ore-mine surveying and some construction work.

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[1] Memoir prepared by B. B. Van Houten, Esq., West Palm Beach, Fla.

From August, 1906, to January, 1907, Mr. Young was a Draftsman, and from January to November, 1907, an Assistant Engineer in the Maintenance-of-Way Department, of the Southern Railway Company, at Birmingham, Ala., on the location and construction of spur lines, etc. He also had local charge of the construction of a freight terminal at Atlanta, Ga.

From 1907 to May, 1912, he was with the United States Engineer Department, until August, 1909, as Surveyor on river surveys, then Junior Engineer on surveys for two storage reservoirs on the Tallapoosa River, in Alabama, part of the time in charge of party, several months on map work, and, later, in local charge of the construction of Lock No. 4 and on channel regularization, Coosa River, at Rome, Ga., Lincoln, Ala., etc.

Mr. Young was in Florida, from June, 1912, to 1916, with the Savannah Engineering and Construction Company, in charge of the construction of reinforced concrete plants (costing approximately $900 000) for drying and storing phosphate rock.

From 1916 to 1923, he was with the Southland Steamship Company (successors to the Savannah Engineering and Construction Company) in charge of construction of about eighteen dams and locks in the main drainage canals of the Florida Everglades, except that in 1918, he was called to Savannah, Ga., where he spent several months superintending the building of ships then under contract from the Federal Government.

From 1923 to July, 1926, he was engaged, for himself, in the development of business and residential properties in Palm Beach County, Florida.

From July, 1926, to May, 1930, he was Consulting Engineer and in charge of the design and construction of ocean beach and boulevard protective bulkheads and groynes for the Town of Palm Beach, at a cost of about $700 000, and from May, 1930, to 1933, he was Consulting Engineer for the Town of Palm Beach, and for many individuals particularly with regard to their ocean-beach protection needs. During this time he secured a Government patent on a special type of groyne for shore protection—a type which both he and others have used most successfully.

Although stricken in the very prime of his manhood with a malady from which very few recover, Mr. Young never gave up but, on the contrary, put up his usual stubborn and determined fight to regain his health, just as throughout his whole professional career he had ever fought to overcome all obstacles in his way and to gain that high degree of success which is measured by the confidence and love of one's fellow men. He was a man of strict integrity and square dealing—the same yesterday, to-day, and to-morrow. As he expressed it, his code was "when I undertake a piece of work, whether it be for others or myself, my sole thought is to successfully complete that work with the least possible delay and at a minimum cost and thus gain the confidence and approval of my clients and the satisfaction of my own conscience." He died as he had lived, fighting for the life which he believed should be his for years to come—his to use for the benefit of his dependent loved ones and all mankind just as he had used the years which had gone before.

Mr. Young was member and Director of the American Shore and Beach Preservation Association, and the following is quoted from a letter written by its President:

> "We were all very fond of Mr. Young. His efforts in the work we were undertaking and his presence at our meetings all caused us to become greatly attached to him. He played a large part in the formation of the organization and was one of those most active in enabling it to serve the public not only in his own community but of the nation. It must be satisfying to you to know that his life was not an empty one but one full of service and usefulness not only to his community, State, and Nation, but to those with whom he came in contact personally."

One short extract, as quoted subsequently, from resolutions passed by The Engineers Club, of Palm Beach County (of which club Mr. Young was a Charter Member and Director), very aptly expresses the feelings of deep respect and high esteem of those who came in contact with him:

> "* * * that in the death of Oliver E. Young the Engineers Club of Palm Beach County has lost a wise counsellor, and each member of the club a true and sincere friend, whose presence will be missed and whose memory will be cherished."

On October 26, 1915, Mr. Young was married to Margaret Mary Southard, of Lakeland, Fla., who, with two children, Mary Elizabeth and Oliver Earle, Jr., survives him.

Mr. Young was elected a Junior of the American Society of Civil Engineers on June 6, 1911, and an Associate Member on March 4, 1913.

THEODORE BRAND, Jun. Am. Soc. C. E.[1]

DIED NOVEMBER 12, 1933

Theodore Brand, the son of Samuel and Catherine Brand, was born at Taylor, N. Dak., on June 20, 1908. He obtained his grade and preparatory education at the Peddie School, at Hightstown, N. J., from which he received his letters in athletics, and where he was a member of Cum Laude, an Honorary Scholarship Society, and of the Peddie House Committee.

Mr. Brand entered Yale University, at New Haven, Conn., with the Class of 1930, and the following year he took up Civil Engineering at the Sheffield Scientific School. For a year, between his Junior and Senior years at Sheffield, he was employed as Acting City Engineer, in Dickinson, N. Dak. In 1931, he was graduated from the Sheffield Scientific School, Yale University, with the degree of Bachelor of Science in Civil Engineering. While at the University, he was prominent in athletics, a member of the Chi Phi Fraernity, and President of the Student Chapter of the American Society of Civil Engineers.

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[1] Memoir prepared by Samuel Brand, Esq., Taylor, N. Dak.

In October, 1931, Mr. Brand was appointed Junior Engineer in the Office of the United States Bureau of Reclamation at Denver, Colo., and until his death on November 12, 1933, he was employed on the Boulder Dam work. He had been recommended in Washington, D. C., for promotion to a higher classification and salary.

Mr. Brand was a Second Lieutenant in the Coast Artillery Reserve Corps and was working for promotion in that branch of the Service. He was also a member of the Masonic Order.

In his work as City Engineer in Dickinson, Mr. Brand showed remarkable ability and qualities of rare leadership and resourcefulness.

From Denver, J. L. Savage, M. Am. Soc. C. E., Chief Designing Engineer of the U. S. Bureau of Reclamation, writes of him:

> "It was my good fortune to follow Ted's work closely for several months, and I know from this association what a splendid man and engineer we have lost. He was one of the most outstanding of the younger men we employed when the expansion was made for the Boulder Dam work."

The following is quoted from a letter received from Dr. M. A. Brannon, *ex*-Chancellor of the University of Montana:

> "He was so fine. Such a high quality mind and man, that it was a real inspiration to have known him intimately as we did while he was with us in Dickinson. He was full of courage in every one of his relationships. He was truly heroic. Hence we admired, trusted, and loved him more than I can tell. He enriched every contact in life and in his difficult and important profession. Society has lost a great, a truly great son. We are deep in grief for you parents, for society, and for ourselves."

Mr. Brand's sterling manhood and rare qualities of character won him friends of all classes and of all ages. He had proved his outstanding ability and capability as an engineer, and he had the brightest prospects ahead of him for a more successful career, when he passed away. His great life ambition was "to know something of engineering, through hard work, study, and experience."

He is deeply mourned by his parents who survive him, by teachers and classmates from Peddie and Yale, by his co-workers, and by hosts of friends. Those who met him admired him, and were impressed by his strong, fine, manly character. Those who knew him, loved him.

Mr. Brand was elected a Junior of the American Society of Civil Engineers on October 26, 1931.

NATHAN GINSBURG, Jun. Am. Soc. C. E.[1]

DIED NOVEMBER 24, 1932

Nathan Ginsburg, the son of Albert and Florence Ginsburg, was born at East Boston, Mass., on May 15, 1904. He attended the public schools of Boston, and was graduated from the East Boston High School in June, 1920, with the highest scholastic honors of his class. He was not only a fine scholar, but a good athlete; he was a leader of his class on the track and in baseball. Several of the track records established by him remained unbroken for a number of years.

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[1] Memoir prepared by Albert Adelman, Esq., Dorchester, Mass.

In September, 1921, Mr. Ginsburg entered the Massachusetts Institute of Technology, at Cambridge, Mass., where he enrolled in the Civil Engineering Course, specializing in hydraulics. He established an excellent scholastic record in spite of his youth. In addition, he was awarded a commission in the Reserve Officers Training Corps. Throughout his college years he maintained an active interest in the athletic program, participating in baseball and tennis.

Mr. Ginsburg began his professional career in July, 1924, in the employ of the Boston Elevated Railway Company. He was engaged first as a Draftsman and, later, in checking the stability of old bridges under new loading conditions. While he realized that his work was interesting and probably would lead to a position of greater importance, he was not satisfied because his mind was set on the hydraulic field for which he had prepared.

He resigned from his position with the Boston Elevated Railway Company and, after working for short periods with the Massachusetts Highway Department and the Norfolk County Engineering Department, he finally entered the employ of Stone and Webster, Incorporated. Here, he was again disappointed in his desire for hydraulic design experience and left this position in May, 1928, to accept an engagement with the New England Power Construction Company. In June, 1930, he again entered the employ of Stone and Webster, Incorporated, as a Hydraulic Designer. His unexpected death brought to an untimely end a career which gave great promise.

Mr. Ginsburg was a member of the Boston Society of Civil Engineers and of the Society of American Military Engineers.

He was not married, and is survived by his mother and father, two brothers, and two sisters.

Mr. Ginsburg was elected a Junior of the American Society of Civil Engineers on October 14, 1930.

THOMAS JEFFERSON HAYDEN, JR., Jun. Am. Soc. C. E.[1]

Died December 9, 1933

Thomas Jefferson Hayden, Jr., was born at Orangeburg, S. C., on July 23, 1906. He was graduated from Clemson College, the South Carolina Agricultural and Mechanical College, at Clemson College, S. C., in 1927, with the degree of Bachelor of Science in Civil Engineering.

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[1] Memoir prepared by Wyllys H. Taylor, M. Am. Soc. C. E.

Immediately after his graduation, Mr. Hayden entered the Engineering Department of the Piedmont and Northern Railway Company, as Structural Draftsman in the Charlotte, N. C., Office. In March, 1928, he joined the Chemical Construction Corporation of Charlotte, as Engineer and remained with this Company until March, 1930, when he became associated, as a Structural Draftsman and Designer, with the W. S. Lee Engineering Corporation, of Charlotte. From October, 1931, to April, 1932, he did concrete designing for the Champion Fiber Company, of Canton, N. C., and from August, 1932, to March, 1933, he was engaged in cost calculations for the Slater Manufacturing Company, of Slater, S. C.

From April, 1933, until his death in December, 1933, he was engaged with the United States Coast and Geodetic Survey in triangulation surveys in Western North Carolina and Virginia. While occupied in this work, and in line of duty, he met with an automobile accident on December 6, that resulted in his death on December 9, 1933, in the Rockingham Memorial Hospital, at Harrisonburg, Va.

Mr. Hayden was a First Lieutenant of Infantry in the Officers Reserve Corps, and was a member of the 324th Infantry with Headquarters in Greensboro, N. C. He took a keen and active interest in the work of the Corps, and was making steady and regular progress in it. He was a Master Mason in Excelsior Lodge No. 261, of Charlotte, and a member of the Dilworth Methodist Episcopal Church in the same city.

He was unmarried and is survived by his parents, Mr. and Mrs. Thomas Jefferson Hayden, and a sister, Margaret Hayden, all of Charlotte, N. C., and by a sister, Mrs. Wyllys H. Taylor, of Slater, S. C.

His bright, cheerful disposition, and pleasing personality endeared him to his associates, while his keen mind and sterling character would have undoubtedly assured him success in his chosen profession had his life been spared.

Mr. Hayden was elected a Junior of the American Society of Civil Engineers on October 26, 1931.

FRANK FIRTH HORD, Jun. Am. Soc. C. E.[1]

DIED JANUARY 5, 1934

Frank Firth Hord was born on June 28, 1904, at Germantown, Philadelphia, Pa., the son of the Rev. Arnold Harris Hord, Registrar of the Protestant Episcopal Diocese of Pennsylvania, and Annie Robb (Firth) Hord.

His secondary education was obtained at the Episcopal Academy of Philadelphia. From there, he went to Yale University, at New Haven, Conn., and was graduated in Civil Engineering from Sheffield Scientific School in 1928.

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[1] Memoir prepared by J. Jervis Vail, M. Am. Soc. C. E.

Mr. Hord was of a family attached to the Pennsylvania Railroad, his grandfather, whose namesake he was, having been President of the Erie and Western Transportation Company, a part of the Pennsylvania System, and his great-grandfather, Thomas T. Firth, was a Treasurer. Upon graduation from Sheffield, Mr. Hord naturally turned to the Pennsylvania Railroad. Being an ardent lover of outdoors, his inclination was to the Construction Department.

He was assigned to work under the Chief Engineer in Northern Jersey and in New York, N. Y., beginning his service in November, 1928. He was very happy in his work. His experience was a rich one, and he attained a responsible position almost at once, on notable construction projects. His first engagement was on the joint Lehigh Valley-Pennsylvania Bridge, over Newark Bay, nearly a mile in length, with a lift span of 330 ft, then the longest and heaviest of its type. On the completion of this structure, early in 1930, Mr. Hord was assigned to the building of two railroad bridges over the Hackensack River at Jersey City, N. J. The foundations were of extreme depth, of the pneumatic form of construction. Here, also, a lift span of maximum size was erected, a counterpart of the one at Newark Bay. Foundations for a highway bridge, a project of the Highway Commission of the State of New Jersey, was included in the work. The bridges were completed and placed in service on November 2, 1930. The final operation called for the floating into position without delays to trains of a truss of 245-ft span, the erection between trains of a 115-ft girder span over an abandoned channel, and the placing thereon of precast concrete slabs, the dredging out of abandoned trestle bridges, and the opening of a new channel for navigation.

His later work took Mr. Hord to the Pennsylvania Terminal, in New York City, where the station was underpinned and new columns were placed, in preparation for the building of a post-office over the railroad tracks. Prior to his death, he was engaged on the heavy construction project at Newark, N. J., embracing very extensive bridges and viaducts and a large passenger station.

In all these works Mr. Hord had an important part. His brilliant mentality, his energy, his complete devotion to the interest of the work, his modesty and friendliness to all, from the laborer up, made him beloved and admired by those who were so fortunate as to know him. He was a great engineer in the making and a perfect gentleman. Above all, his character was unspotted and his ideals were of the highest.

Mr. Hord was active in the Engineer Officers Reserve Corps, holding a commission as Second Lieutenant. His club memberships embraced the Society of American Military Engineers, the Yale Engineering Association, the Yale Clubs of New York and Philadelphia, and the Society of Colonial Wars.

In addition to his parents, he is survived by a brother, William Taliaferro Hord, and a sister, Annie Lloyd Hord.

Mr. Hord was elected a Junior of the American Society of Civil Engineers on April 15, 1929.

FRANK PIERCE MESERVE, JR., Jun. Am. Soc. C. E.[1]

DIED JULY 14, 1933

Frank Pierce Meserve, Jr., the son of Mr. and Mrs. Frank Pierce Meserve, was born in Redlands, Calif., on September 19, 1905. He obtained his early education in the public schools of Redlands. In 1923, he matriculated in the California Institute of Technology at Pasadena, Calif., where he specialized in Highway Engineering and Hydraulics. He was graduated from the Institute in 1927 with the degree of Bachelor of Science in Civil Engineering.

From the time of his graduation until his death, Mr. Meserve was continuously employed in the Los Angeles County (California) Road Department. Starting as a Draftsman, his ability brought rapid advancement to the design of reinforced concrete hydraulic and highway structures, and to the supervision of their construction. He was also engaged in a study and analysis of the traffic flow on highways in Los Angeles County, a work which was uncompleted at the time of his death. Desiring a wider experience in his chosen field of work, he applied for a position with the United States Bureau of Public Roads. Notification of his appointment as Chief District Engineer was received two weeks after his death.

Mr. Meserve combined the academic with the practical in his engineering work. From September, 1929, until June, 1933, he was a member of the Faculty of the School of Government in the Civic Center Division of the University of Southern California, teaching courses in Mathematics, Highway Engineering, and Reinforced Concrete. He was also a student in the School of Government, working with the intention of taking the degree of Master of Science in Public Administration. He took an active interest in the local affairs of the Society and of the City and County Engineers Association. He was a First Lieutenant in the Coast Artillery Corps Reserve and had obtained the "Expert" rating on the pistol range.

A quiet, unassuming personality drew to Mr. Meserve a wide circle of friends. His willingness to assume responsibility and his ability to secure results made him extremely valuable to his employers. All agree that his untimely death cut short a career which had promise of going far in the practicing and administrative fields of engineering.

On June 21, 1930, Mr. Meserve was married to Clodie Gaudin, at Los Angeles, Calif. He died on July 14, 1933, after a comparatively short illness, of a malignant growth of the intestine and stomach. In addition to his widow and his parents, he is survived by two brothers, Harold Meserve, of Los Angeles, and Jack Meserve, of San Bernardino, Calif., and one sister Helen Meserve, of Redlands.

Mr. Meserve was elected a Junior of the American Society of Civil Engineers on October 1, 1928.

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[1] Memoir prepared by Henry R. Freeman, Civ. Engr., Los Angeles County Road Dept., Los Angeles, Calif.

SAMUEL JOSEPH GARGES, Affiliate, Am. Soc. C. E.[1]

Died November 30, 1933

Samuel Joseph Garges was born in Washington, D. C., on October 23, 1880, the son of William H. and Mary (Mullen) Garges. He received his education in the public schools of Washington, and was graduated from the Eastern High School in 1898.

Following his graduation, Mr. Garges went to New York, N. Y., with his uncle, Andrew Mullen, an engineer and contractor, in whose office he began the study of engineering. After a year in his uncle's office, he was employed in 1899 as Assistant Engineer by the National Contracting Company, during the construction of the Sixth Avenue Underground Trolley System from 59th Street to Canal Street, New York City.

In 1901, Mr. Garges went with the Barber Asphalt Company which was engaged in building streets in New York City. Shortly afterward, he was transferred to Michigan as Superintendent in charge of the Company's activities in that State, planning, promoting, and estimating contracts for streets, sidewalks, roads, sewers, etc. Mr. Garges remained in Michigan about five years and, in 1907, became District Manager for the Cleveland Trinidad Paving Company. He was stationed at Tulsa, Okla., in charge of its business in Oklahoma and Texas, and was engaged in planning, promoting and estimating, and constructing roads, streets, sewers, sidewalks, and street railway work.

His subsequent activities were, as follows: In 1908 and 1909, he was Eastern Engineer for the Standard Asphalt and Rubber Company. As such, he was engaged in estimating, promoting sales, constructing, and planning for projects for which the Company's asphalt products were used, such as in roads, etc., and in water-proofing, bridges, subways, buildings, etc. In 1909, Mr. Garges was made General Manager by the Eastern Asphalt Company. In this position he was engaged in all kinds of street and road promotion and construction.

In 1917, he was commissioned a Captain in the Construction Division of United States Army, and was stationed at Camp Meade, in Maryland, where as Engineer, he had charge of appraisals, the clearing of farm lands, the construction of buildings, camp layout, etc.; at Curtis Bay, Maryland, as Progress and Assistant Engineer on the construction of the $15 000 000 Ordnance Depot; at Fox Hills Hospital, Staten Island, New York, as Assistant in charge of the layout of the hospital grounds, and the construction of roads, railways, buildings, etc.; and at the Proving Grounds, at Elizabeth, N. J., in charge of all construction.

From 1919 to 1922, Mr. Garges served as Commissioner of Public Works and Engineer, for the City of New Rochelle, N. Y. As such, he had charge of plans for zoning and served as Chairman of the Chamber of Commerce Committee on Town Planning. He was also the originator of the large sewerage system which has since been constructed in New Rochelle.

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[1] Memoir compiled from information on file at the Headquarters of the Society.

Mr. Garges was appointed General Superintendent of the William P. McDonald Construction Company, of New York City, in 1922 and, during that year and 1923, he was engaged in planning, estimating, and building sewers, streets, roads, etc., in New York. In 1924 and 1925, he served as General Manager for the Arthur A. Johnson Company, New York City, in planning, estimating, and building sewers, streets, roads, etc., in various parts of New York State. In 1926, he was made Engineer for the Sanitation Corporation, New York City, and was engaged in problems of city sanitation, such as sewage disposal, collection and incineration of city wastes, and the purification of water supplies.

He served as Chief Engineer of the Sheet Concrete Pavement Corporation of America in 1928, and, in 1929, he was Associate Engineer with the late George A. Johnson, M. Am. Soc. C. E., in New York City, engaged in all branches of sanitary engineering, such as refuse disposal, sewage disposal plants, water purification, etc.

In 1930, Mr. Garges was re-appointed Commissioner of Public Works for New Rochelle, which position he held until November, 1931, when he resigned on account of ill-health. He never re-entered actively into business, and passed away at his home in New Rochelle on November 30, 1933.

He is survived by his widow, the former Anna Schnoor, of Niles, Mich., to whom he was married in Chicago, Ill., in 1917, and by a daughter, Barbara.

He was always greatly interested in yachting and sailing and, at one time, was an active member of the New Rochelle Yacht Club. Later, when his health precluded any strenuous activity, Mr. Garges devoted himself to reading and study, and made some wide researches into American genealogies. He was the author of "Street and Road Conditions in New Rochelle"; "History of Drain and Sewer Development in New Rochelle"; "Harbor Development of New Rochelle"; and "Development of Pine Brook Boulevard."

Mr. Garges was a staunch upholder of the value of college training along technical and professional lines, but his own success was due wholly to his mental capacity, his energy and his persistence in study, and the drive to master the problems of the profession he had chosen.

He was a member of the Society of American Military Engineers, the Sons of the American Revolution, and a Veteran of the 22d Regiment (Engineers), New York National Guard. He was also a member of the Wykagyl Country Club, of New Rochelle.

Mr. Garges was elected an Affiliate of the American Society of Civil Engineers on April 19, 1920.

FRANK MAYHEW TALBOT, Affiliate, Am. Soc. C. E.[1]

DIED MARCH 6, 1933

Frank Mayhew Talbot was born on February 1, 1862, in Lubec, Me., the son of John C. and Antoinette (Wass) Talbot, of East Machias, Me., and the grandson of Judge John C. Talbot, a noted jurist, civic and religious leader, and one of the most noteworthy citizens of East Machias. His father, like his grandfather, was also a successful lawyer and a prominent leader in the civic affairs of Eastern Maine. The genealogy of the Talbot family in America dates back to 1677 when Peter Talbot landed from England on the shores of what was then Eastern Massachusetts.

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[1] Memoir prepared by John C. Talbot, Esq., New York, N. Y.

Frank Mayhew Talbot attended the local schools of East Machias, and was graduated from Washington Academy in 1878 at the age of sixteen. Endowed with a spirit of determination to make his own way in the world and be independent, he left the homestead and started for Minneapolis, Minn., with his railroad fare and a parting gift of a $20 gold piece. Here, he commenced his career in the dry goods business rising rapidly to the position of an expert accountant.

Having a natural desire to accomplish greater achievements, Mr. Talbot branched out into the railroad construction business, starting as a bookkeeper with the Arthur McMullen Company, of New York, N. Y., which Company had the contract for the building of the car shops for the Northern Pacific Railway Company, outside Tacoma, Wash. It was during this work, in 1889, that Mr. Talbot was seriously injured in a railroad accident necessitating the amputation of his right leg below the knee. He carried on bravely in spite of this terrific handicap to success in life. Continuing with the Arthur McMullen Company, he worked on the foundation for the Kingsbridge, in 1893, in New York City, and the building of the Croton Dam and the Holyoke (Mass.), Dam in 1895 and 1897, until he returned to Minneapolis in 1899 to become Superintendent for the C. F. Haglin Company on the construction of the Peavey concrete grain elevator at Duluth, Minn. This was the first concrete grain elevator built in America. Following this work, he had charge of the construction of similar grain elevators at Valley City, N. Dak., and at Minneapolis, all of which are in use to this day.

In 1906, Mr. Talbot returned to the Arthur McMullen Company in New York City, and had charge of the construction of the concrete abutments and retaining walls for the over-head crossing of the Pennsylvania Railroad and the Lackawanna Railroad in the Newark Meadows, in New Jersey. In 1908, he supervised the lining of the Bergen Hill Tunnel with concrete, for the Lackawanna Railroad Company. In connection with this latter work he developed an ingenious method for placing concrete on top of an arch, which he patented.

In 1910, Mr. Talbot organized the F. M. Talbot Company of which he was President, and E. J. Maleady, Secretary and Treasurer, retaining Lincoln Bush, Past-President, Am. Soc. C. E., former Chief Engineer of the Lackawanna Railroad Company as Consulting Engineer, and started in the railroad contracting business. Mr. Bush was subsequently elected Vice-President of the Company. The first work of this Company consisted of the building of the Hayes' Creek abutment, at White Haven, Pa., in 1910, for the Lehigh Valley Railroad Company. Following this job, the Company was successful in being awarded the contract for the double-track concrete arch bridge over the Delaware River, at Yardley, Pa., for the Philadelphia and Reading Railroad Company.

In 1912, the Lackawanna Railroad Company commenced the building of a new 20-mile cut-off from Clark's Summit to Halstead, Pa. involving the construction of the Tunkhannock Concrete Viaduct, at Nicholson, and the Martin's Creek Viaduct, at Kingsley, Pa. Mr. Talbot was personally successful in being awarded the contracts for both these viaducts. The Nicholson job was done under the name of Flickwir and Bush, Incorporated, and the Kingsley work under the name of the F. M. Talbot Company. The Tunkhannock Viaduct was approximately forty-eight months under construction, being the largest railroad concrete arch bridge in the world at the time. It spanned a valley over part of the Town of Nicholson, between two high hills, at an elevation of 240 ft. It was 2 375 ft long, and consisted of ten 180-ft and two 100-ft reinforced concrete arches. The entire railroad cut-off eliminated about 20 miles of severe grades, thereby reducing the running time of fast passenger trains by approximately 20 min, and cost from $12 000 000 to $15 000 000.

At various times, from 1915 to 1925, Mr. Talbot's Company built the overhead street crossings and retaining walls for the Lackawanna Railroad Company through Orange, Ampere, and part of East Orange, N. J., in connection with the elimination of grade crossings through these cities, and other similar projects, such as those at Passaic, Mountain View, and Convent Station, N. J.

During the World War, the Company was awarded the contract for strengthening the underpinning of the center pier of the Passaic River Drawbridge, of the Pennsylvania Railroad Company, at Market Street, Newark, N. J., necessitated by the deepening of the channels in the river by Government dredging operations. This was an extremely difficult and treacherous piece of work, which was completed without causing any interference to the drawbridge, or without delaying a single train on one of the most important arteries for the transportation of troops and war materials entering Metropolitan New York.

Mr. Talbot practically retired from active construction work in 1925, completing one or two smaller jobs after this date, although he continued as President of the F. M. Talbot Company until his death.

He was a member of the Railroad Club of New York. He was also a member of Montclair Lodge No. 144, F. and A. M., Montclair Chapter No. 54, R. A. M., and a Knight Templar. During the last twenty-seven years, he had made his home in Glen Ridge, N. J.

He is survived by his widow, Loretta Townsend, of Minneapolis, a son, John C. Talbot, and one grand-daughter, Jeanne Talbot, of Montclair, as well as two sisters, Esther B. Talbot and Mary T. Woodman, of Minneapolis. He was buried in the Talbot plot in Lakewood Cemetery, Minneapolis, Minn.

Mr. Talbot was a man of rare judgment and executive ability, kind and sympathetic in all his dealings with men, and endowed with an ingenious mind which was never dormant. He was ever ready to help and assist the man who was down, and his acts of charity, rendered in his own quiet way, will long be remembered.

His untimely death on March 6, 1933, from pneumonia, marked the passing of another of the prominent railroad contractors whose everlasting landmarks remain as a tribute to their courage and ability in promoting the advance of the railroad systems in America.

Mr. Talbot was elected an Affiliate of the American Society of Civil Engineers on May 4, 1909.

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